MONTANA
DEPARTMENT OF HIGHWAYS

HYDRAULICS MANUAL

SECTION 4
DESIGN

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4.1 CULVERTS

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Introduction

Designing highway culverts involves many factors including estimating flood peaks, hydraulic performance, structural adequacy and over all construction and maintenance costs. The procedure for estimating flood peaks was outlined in Section 3 and the method for determining the structural adequacy will be given in Section 4.14. The procedures for determining the hydraulic capabilities will be outlined in this section.

The design method and information presented here are taken from Hydraulic Engineering Circular No. 5, "Hydraulic Charts for the Selection of Highway Culverts", and Hydraulic Engineering Circular No. 13, "Hydraulic Design of Improved Inlets for Culverts". This design procedure is valid for all conventional culverts, such as circular, arch, and oval pipes, both metal and concrete, and concrete box culverts. All such conventional culverts have a uniform barrel cross section throughout. The improved inlet design charts taken from Circular No. 13 apply only to rectangular or circular barrel shapes. Since improved inlet tests were not conducted on either arch or oval barrel shapes, the coefficients, curves, and design procedures cannot be included in this manual at this time. However, the improved inlet design concept is applicable to oval and arch pipe and design curves, etc., will be added to this manual when they become available. Until that time, arch and oval shapes should be considered as alternates and the basic concepts discussed in the Culvert Hydraulics Section should be applied to arch and oval pipes.

Culvert flow capacity is limited either by the culvert entrance conditions or by barrel resistance. The former is designated "inlet control" and the latter "outlet control". When a culvert operates in inlet control, the barrel

will permit the passage of more flow than the inlet, and in outlet control the reverse is true.

If inlet control governs, inlet improvements can result in the need for a barrel size smaller than would be required for a conventional culvert at the same site. The amount of barrel size reduction depends on the site and a subjective judgement regarding the dependability of the design flood estimate and the risk of damage inherent in exceeding the allowable headwater elevation. If the design discharge estimate is not well supported and considerable damage would result if the allowable headwater elevation were exceeded, it may be wise to select a culvert barrel somewhat larger than would be required to accommodate the design discharge. On the other hand, if the design discharge estimate is liberal or well supported by data and analysis or a headwater elevation higher than the allowable would result in little or no damage to the highway or the adjacent property, then the smallest possible barrel size might be selected. Design techniques presented here will enable the designer to evaluate the hydraulic variables and select the most rational design for the particular site.

The general benefits of good culvert design procedures include reduction of upstream flooding and highway damage due to underdesign and lower culvert construction costs by avoiding gross overdesign. If site conditions permit the use of an improved inlet, construction costs may be reduced still further. At times, improved inlets may also be installed on existing culverts with inadequate flow capacity, thus avoiding replacement of the entire structure or the addition of a new parallel structure.

General Design Considerations

There are many factors to consider in culvert design in addition to hydraulic and structural adequacy, many of which are subjective. Following is a discussion of some of the aspects that should be considered in culvert design.

Highway Safety Aspects

Culvert inlets and outlets should be located a sufficient distance from the pavement so as not to present an undue hazard to errant vehicles. Otherwise, suitable restaints should be provided to prevent vehicles from colliding with the inlet structures.

Hydrologic Estimates

The design discharge for a culvert is an estimate, usually made with some recognition of the risk involved or the chance that the discharge will be exceeded. For instance, there is a 2 percent chance that the 50 year flood will be exceeded in any one given year. Or, a structure with a 25 year life expectancy designed for the 50 year flood has a 40 percent chance of experiencing a higher flood during its life. If the frequency analysis is based on short period of flood or streamflow records, the chances of the estimated peak for the design flood being exceeded are much greater.

This creates the necessity of evaluating a culvert's performance through a range of discharges. The risk of damage to the highway or adjacent property due to floods greater than the design discharge may be greater with these culverts than with conventional culverts, as performance may shift to outlet control. The designer should examine the performance of the proposed culvert in outlet control to determine whether or not that performance is acceptable.

Allowable Headwater Elevation

The maximum permissible elevation of the headwater pool of the culvert at the design discharge is termed the Allowable Headwater Elevation. This elevation must be selected by the designer based on his evaluation of many factors, all of which should be well documented. These include highway elevations, upstream development and land use, feature elevations, historical high water marks, importance of the highway, and damage risks. Possible loss

of life and property and traffic delay and interruption should be considered in the damage risk analysis.

Throughout the design process, the designer should remain aware of the consequences of exceeding the Allowable Headwater Elevation. In some situations, such as in rural areas, the damages might be negligible, while in others, exceeding the Allowable Headwater Elevation should definitely be avoided.

Drift and Debris

A frequent objection to the use of culverts instead of bridges and the use of side- and slope-tapered inlet configurations in particular is that problems with drift and debris will increase. If it is anticipated that the drainage basin will contribute a large amount of drift and debris, the debris control design procedures presented in Section 4.15 should be utilized. These procedures are taken from Hydraulic Engineering Circular No. 9, "Debris Control Structures".

Sedimentation

For conventional and improved inlet culverts with their barrels on nearly the same slope as the original streambed, no unusual sedimentation problems are to be expected.

The inlets with FALLS have barrels on a flatter slope than the streambed, which may tend to induce some sedimentation, especially at low flow rates. These deposits will, however, tend to be washed out of the culvert during periods of higher discharge.

Outlet Velocity

Intuitively, it would seem that reducing the size of the culvert barrel would increase scour problems at the outlet due to increased outlet velocities. On the contrary, the outlet velocities for a conventional culvert and a culvert with an improved inlet for the same location and design conditions are essentially the same. When the barrel area is reduced, the flow depth is

increased, and the flow area and velocity remain essentially the same. This fact can be confirmed by reviewing the example problems.

Outlet velocity is simply the discharge divided by the flow area at the outlet. For culverts flowing in inlet control, the depth at the outlet is approximated by assuming the flow approaches normal depth. This depth may be determined by trial and error using a form of Manning's Equation:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

A detailed discussion of Manning's Equation and charts for the solution of the equation for trapezoidal channels are provided in Section 4.61.

For culverts flowing in outlet control, the depth is assumed to be: critical depth when the tailwater depth is less than critical depth; the tailwater depth when it is greater than critical depth but less than the culvert height; or the full culvert height when the tailwater is equal to or greater than the height of the culvert or when critical depth is greater than the height of the culvert.

If scour due to high velocity is anticipated at the outlet end, erosion control measures should be taken as presented in Section 4.8.

Orientation with Stream

Faces for both the side-tapered and slope-tapered inlets should be oriented normal to the direction of flow in the stream and not necessarily parallel with the roadway centerline. By constructing the entrance in this manner, hydraulic performance will be improved and structural design complications reduced. The embankment may be warped to fit the culvert and remain aesthetically pleasing.

Avoiding inlet skew is especially important in multiple barrel culverts. The interior walls, which are neglected in unskewed culverts, may produce

unequal flow in the culvert barrels, reduced performance, and possible sedimentation in some barrels.

Culvert Cost

The total cost of various alternatives should be considered in the final culvert selection. For instance, a slope-tapered installation or a side-tapered inlet with a depression will probably require more excavation than a culvert with its invert near the original stream flowline. If this excavation must be made through rock or other difficult material, it may be more economical to use a side-tapered design, assuming that both designs are hydraulically feasible, even though the barrel size of the slope-tapered culvert may be smaller.

Culvert Length

As previously mentioned, the culvert barrel cost usually far outweighs the cost of the inlet structure. Therefore, if a very long culvert operates in inlet control, opportunities may exist for great savings by using an improved inlet and reducing the barrel size.

Short culverts should also be analyzed for possible cost reductions through the use of improved inlets. Many significant savings have been recorded for these structures, especially in cases where the capacity of an existing culvert was increased by addition of an improved inlet rather than by replacement of the entire culvert.

Fish Passage

Many streams in which culverts are placed contain an active population of game fish and passage of these fish is an important design consideration. Since no specific design procedure has been developed for fish passage, the following publications, which discuss and make recommendations for fish passage, are listed for designer reference. The designer should consult them as needed.

1. Howard E. Metsker, "Fish Versus Culverts (Some Considerations for Resource Managers)", Engineering Technical Report ETR-7700-5, U.S. Forest Service, U.S. Department of Agriculture, July, 1970.

2. Thomas J. McClellan, "Fish Passage Through Highway Culverts (A Field Evaluation)", Federal Highway Administration, U.S. Department of Transportation, 1970.

Culvert Hydrualics

Conventional Culverts

A culvert operates in either inlet or outlet control. Under outlet control, headwater depth, tailwater depth, entrance configuration, and barrel characteristics all influence a culvert's capacity. The entrance configuration is defined by the barrel cross sectional area, shape, and edge condition, while the barrel characteristics are area, shape, slope, length, and roughness. As shown in Figure 4.1, the flow condition for outlet control may be full or partly full for all or part of the culvert length. The design discharge usually results in full flow. Inlet improvements in these culverts reduce the entrance losses, which are only a small portion of the total headwater requirements. Therefore, only minor modifications of the inlet geometry which result in little additional cost are justified.

In inlet control, only entrance configuration and headwater depth determine the culvert's hydraulic capacity. Barrel characteristics and tailwater depth are of no consequence. These culverts usually lie on relatively steep slopes and flow only partly full, as shown in Figure 4.2. Entrance improvements can result in full, or nearly full flow, thereby increasing culvert capacity significantly.

Figure 4.3 illustrates the performance of a 30-inch circular conduit in inlet control with three commonly used entrances: thin-edged projecting, square-edged, and groove-edged. It is clear that inlet type and headwater depth determine the capacities of these culverts. For a given headwater, a groove-

edge inlet has a greater capacity than a square-edged inlet, which in turn outperforms a thin-edged projecting inlet. The performance of each inlet type is related to the degree of flow contraction. A high degree of contraction requires more energy, or headwater to convey a given discharge than a low degree of contraction. Figure 4.4 shows schematically the flow contractions of the three inlet types noted in Figure 4.3.

Improved Inlets

The improvements presented here are inlet geometry refinements beyond those normally used in conventional culvert design practice, such as those discussed above. Several degrees of improvements are presented, including bevel-edged, side- and slope-tapered inlet.

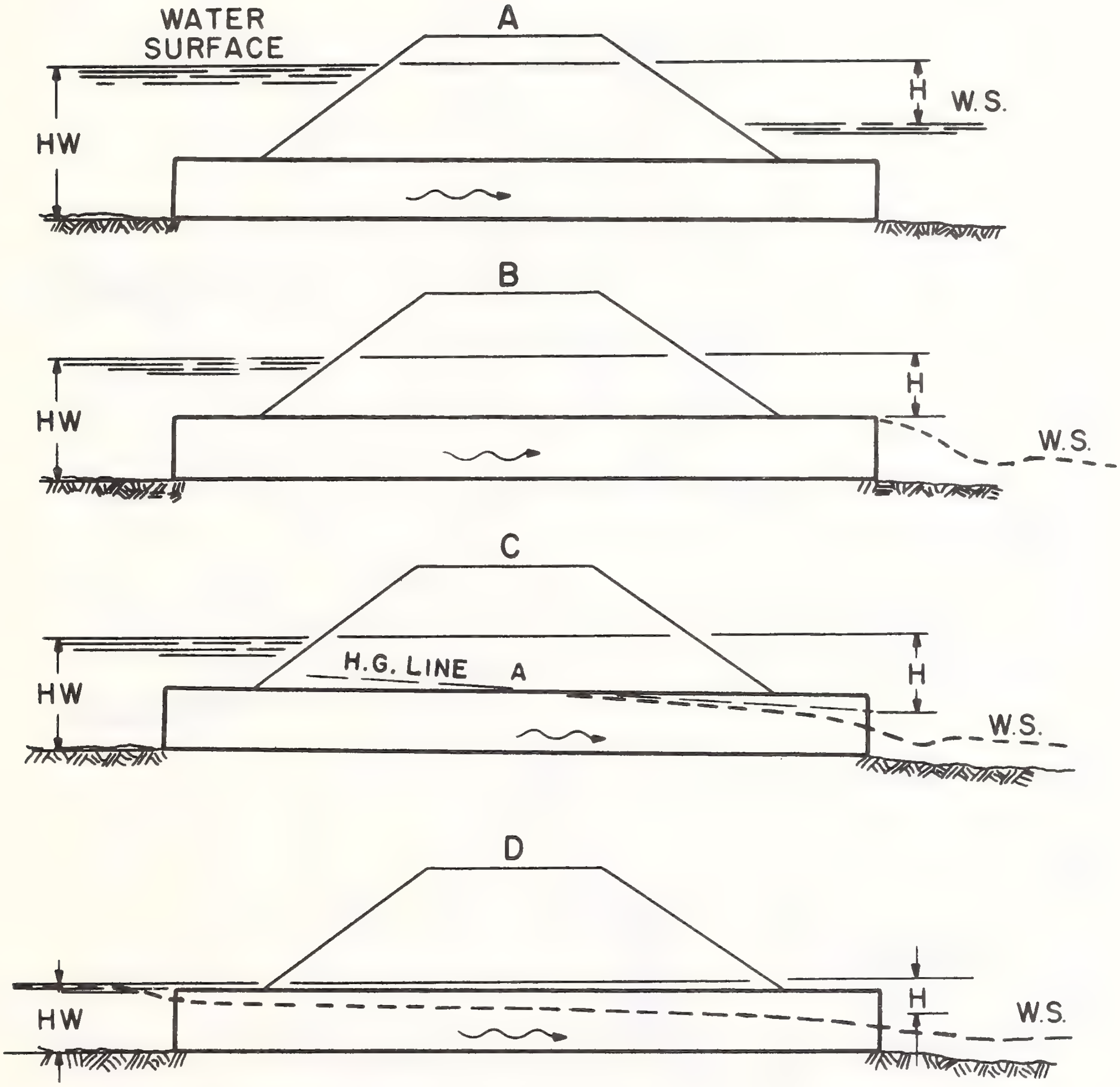
Bevel-Edged Inlets - The first degree of inlet improvements is a beveled edge. The bevel is proportioned based on the culvert barrel or face dimension and operates by decreasing the flow contraction at the inlet. A bevel is similar to a chamfer except that a chamfer is smaller and is generally used to prevent damage to sharp concrete edges during construction.

Adding bevels to a conventional culvert design with a square-edged inlet increases culvert capacity by 5 to 20 percent. The higher increase results from comparing a bevel-edged inlet with a square-edged inlet at high headwaters. The lower increase is the result of comparing inlets with bevels with structures having wingwalls of 30 to 45 degrees.

Although the bevels used herein are plane surfaces, rounded edges which approximate the bevels are also acceptable.

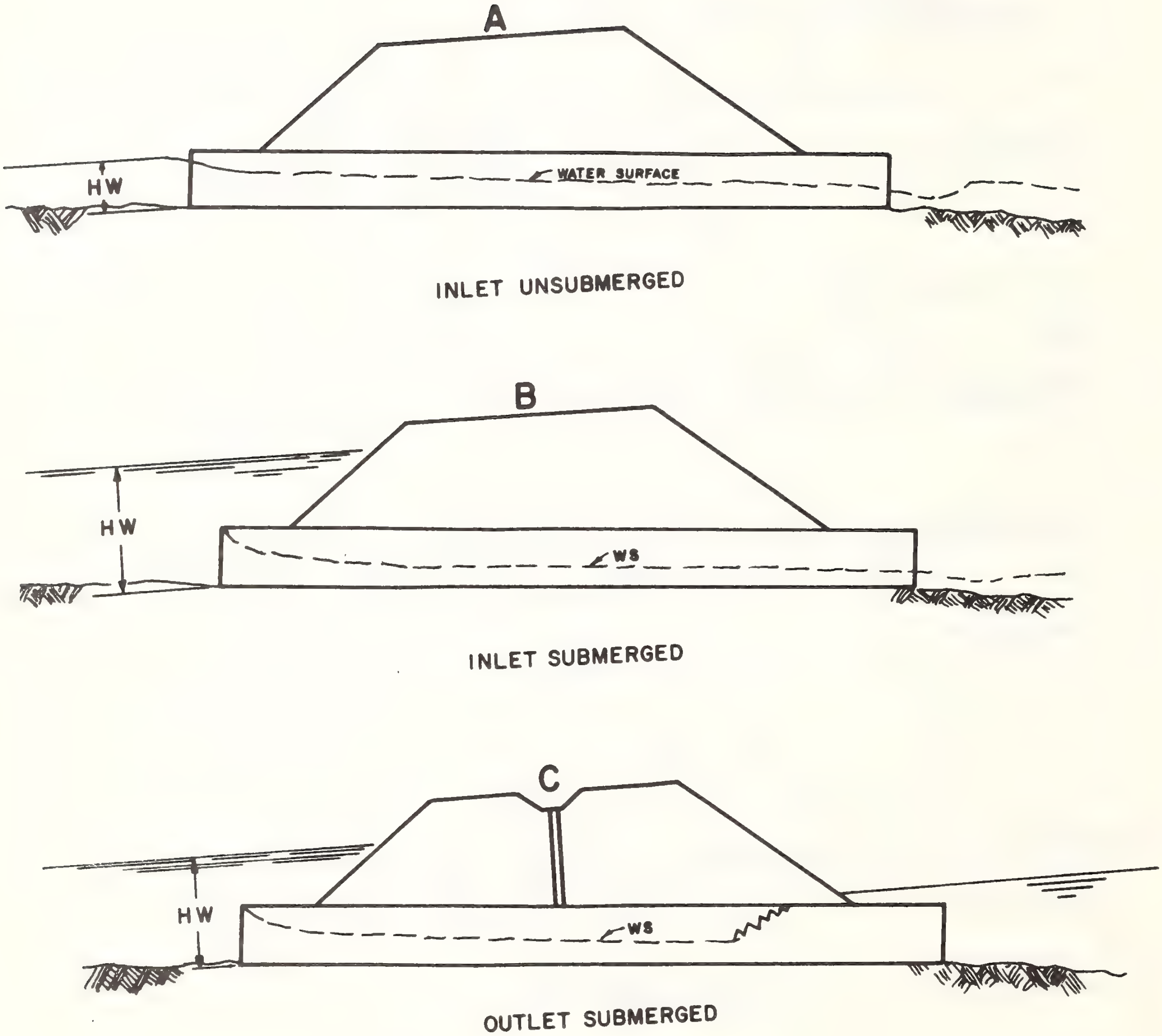
As a minimum, bevels should be used on all culverts which operate in inlet control, both conventional and improved inlet types. The exception to this is circular concrete pipes where the socket end performs much the same as

Figure 4.1



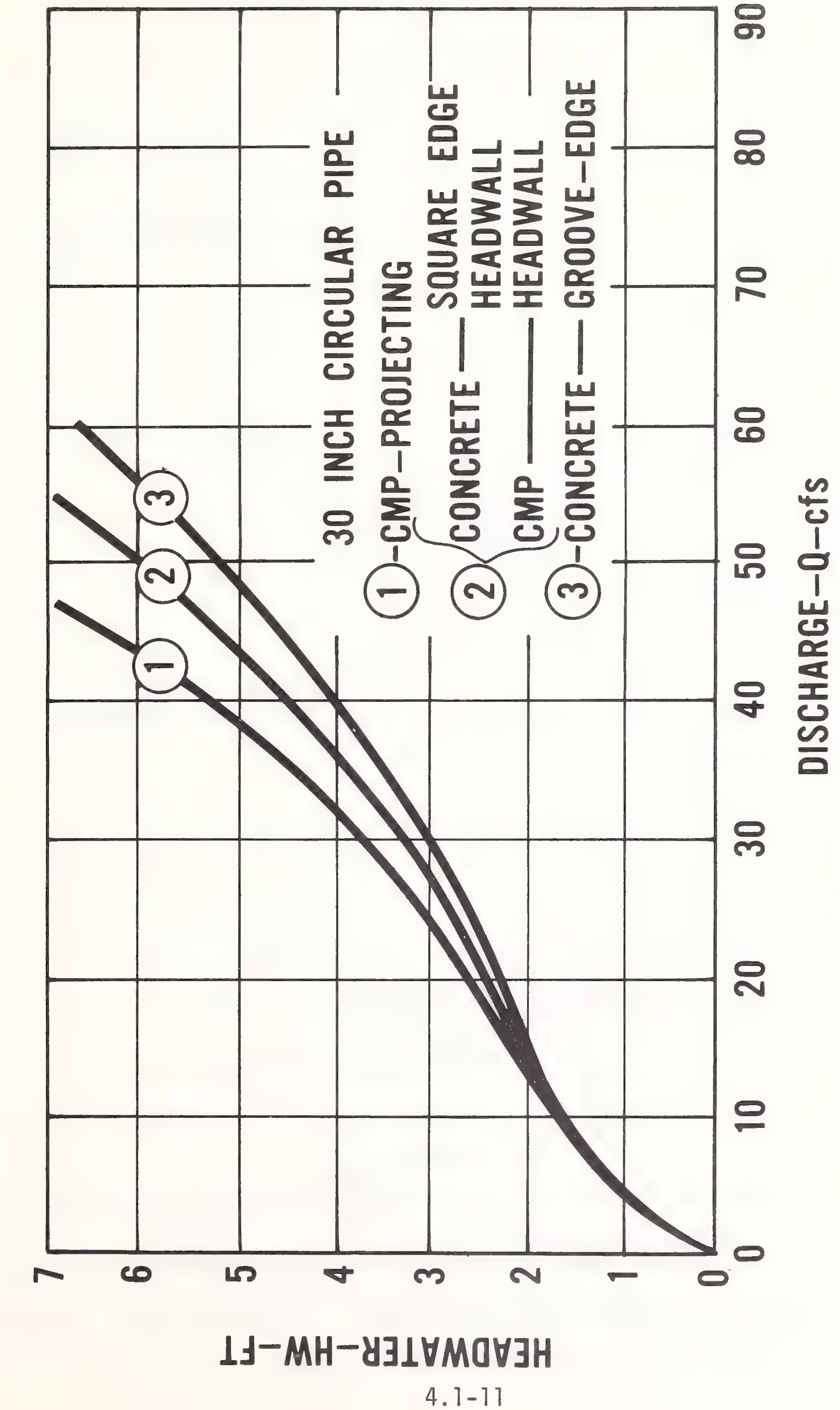
OUTLET CONTROL

Figure 4.2



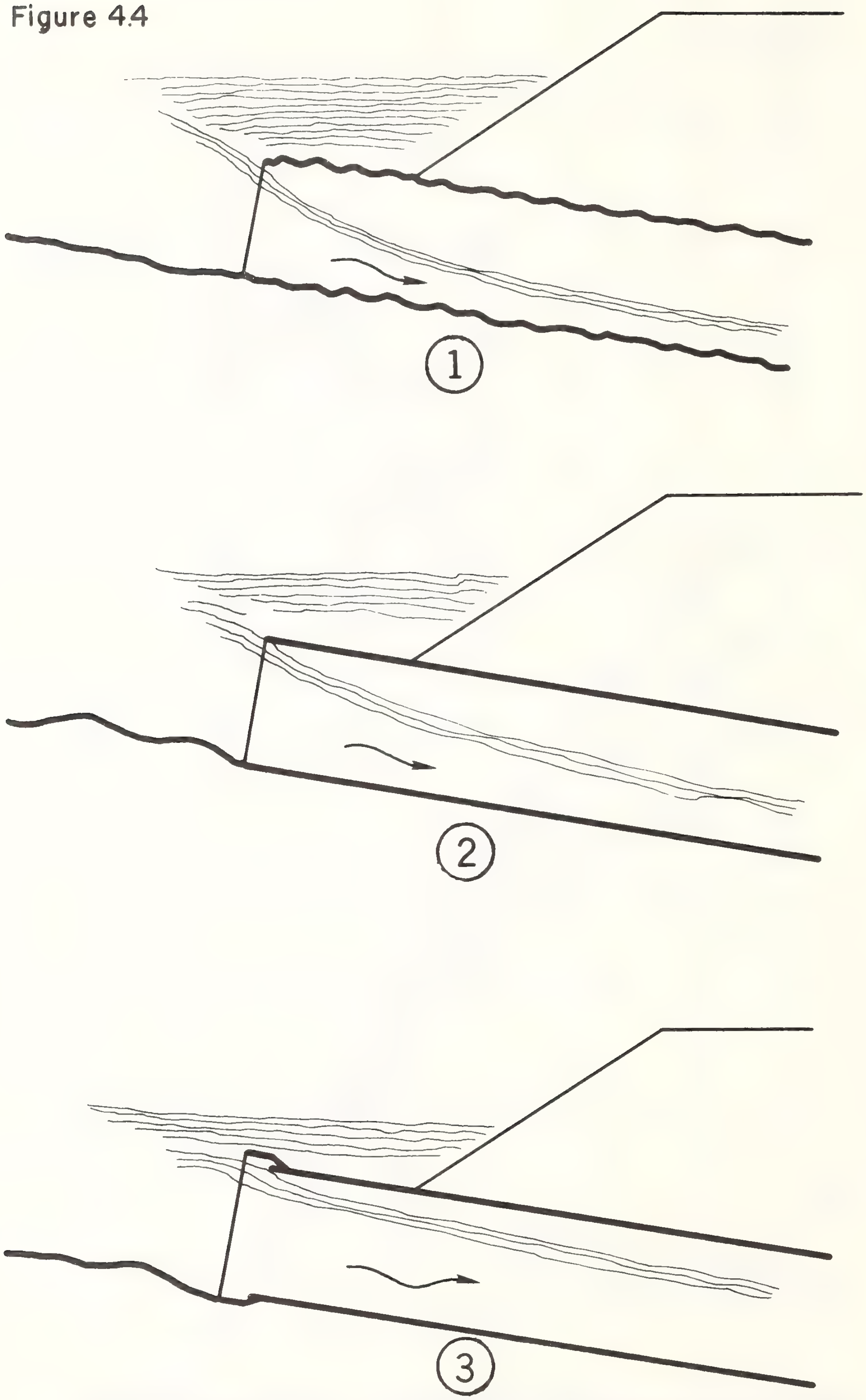
INLET CONTROL

Figure 4.3



PERFORMANCE CURVES INLET CONTROL

Figure 4.4



SCHEMATIC FLOW CONTRACTIONS
FOR CONVENTIONAL CULVERT INLETS

a beveled edge. Examples of bevels used in conjunction with other improved inlets are shown in Figure 4.5 and 4.6. Culverts flowing in outlet control cannot be improved as much as those in inlet control, but the entrance loss coefficient, k_e, is reduced from 0.5 for a square edge to 0.2 for beveled edges. Therefore, it is recommended that bevels be used on all culvert entrances if little additional cost is involved.

Side-Tapered Inlets - The second degree of improvement is a side-tapered inlet (Figure 4.5). It provides an increase in flow capacity of 25 to 40 percent over that of a conventional culvert with a square-edged inlet. This inlet has an enlarged face area with the transition to the culvert barrel accomplished by tapering the sidewalls. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section.

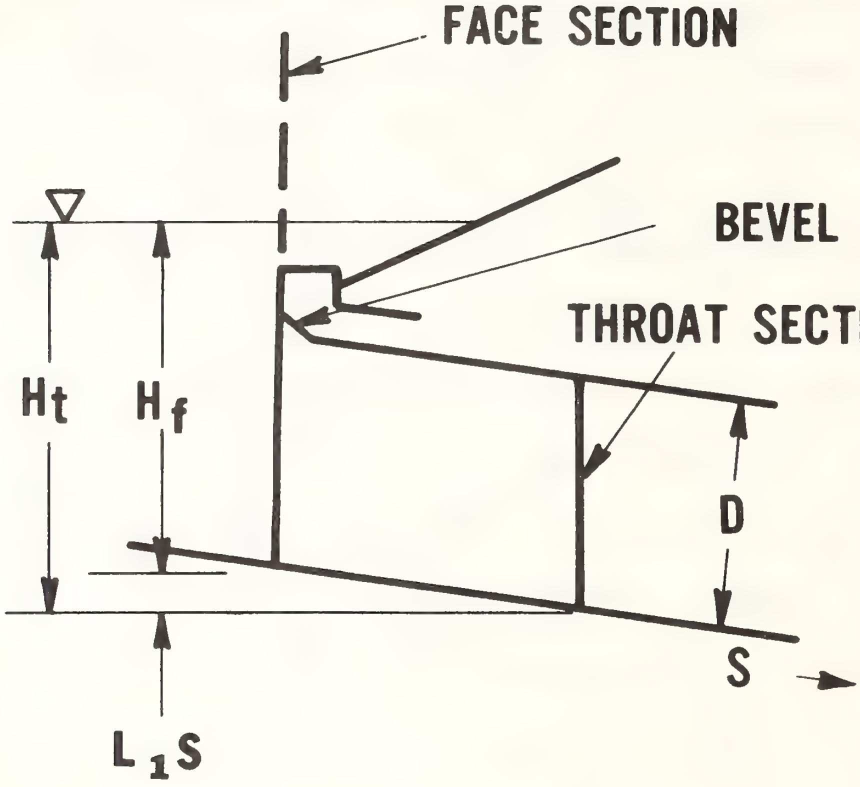
For the side-tapered inlet, there are two possible control sections: the face and the throat. H_f, as shown in Figure 4.5, is the headwater depth based upon face control. H_t is the headwater depth based upon throat control.

The advantages of a side-tapered inlet operating in throat control are: The flow contraction at the throat is reduced; and, for a given pool elevation, more head is applied at the throat control section. The latter advantage is increased by utilizing a slope-tapered inlet or a depression in front of the side-tapered inlet.

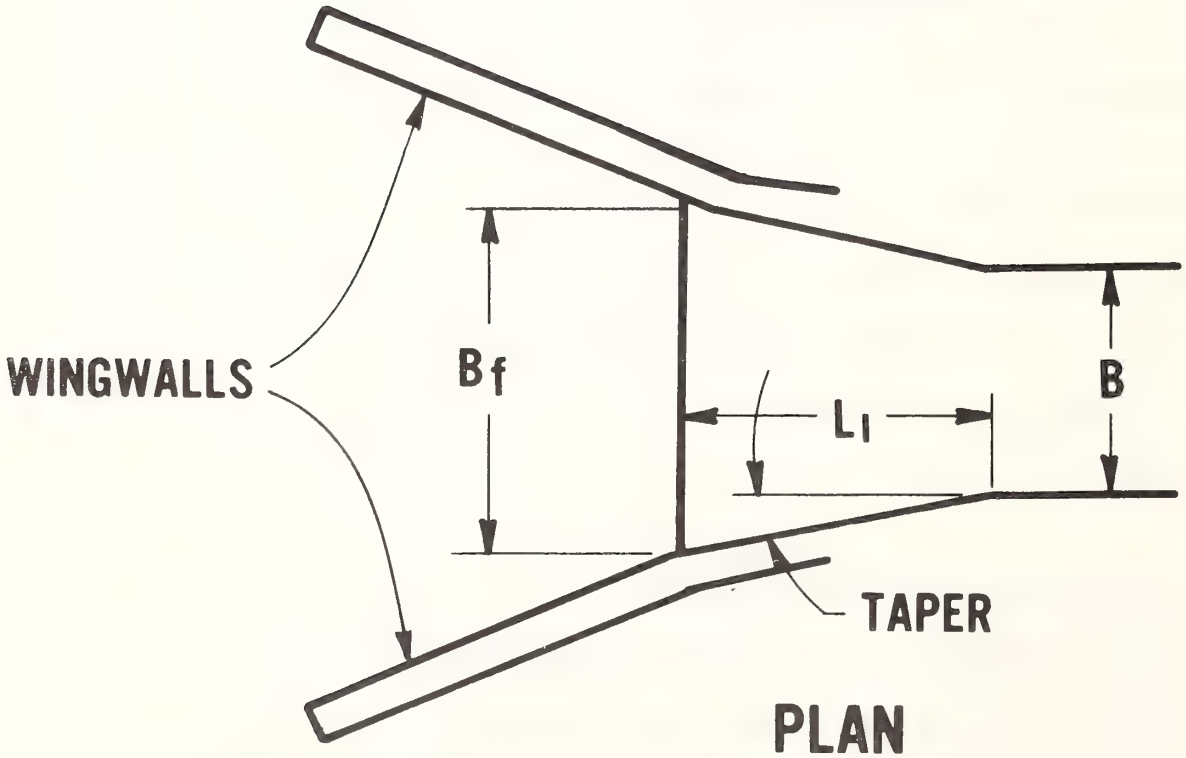
Slope-Tapered Inlets - A slope-tapered inlet is the third degree of improvement. Its advantage over the side-tapered inlet without a depression is that more head is available at the control (Throat) section. This is accomplished by incorporating a FALL in the enclosed entrance section (Figure 4.6).

Figure 4.5

SIDE - TAPERED INLET



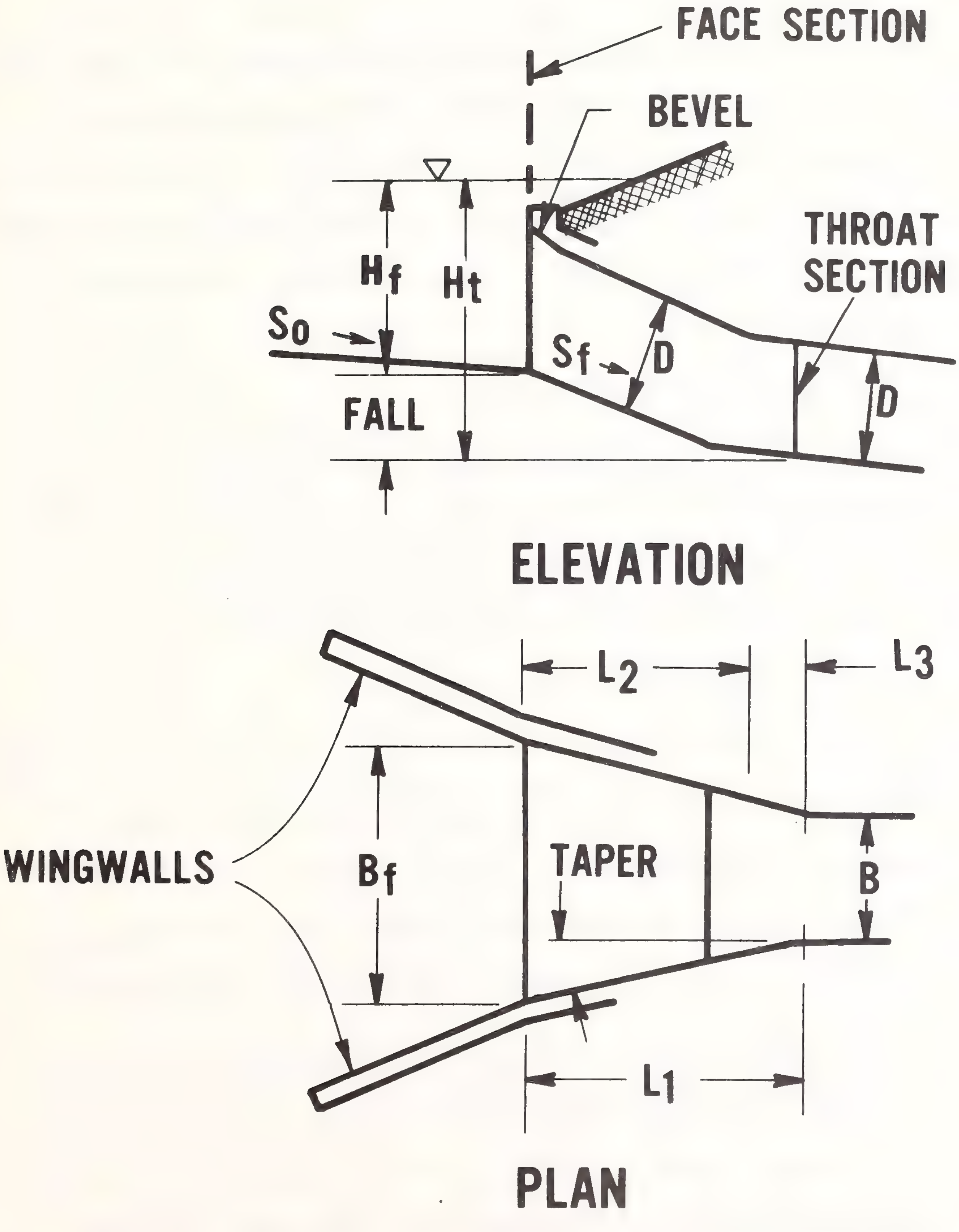
ELEVATION



PLAN

Figure 4.6

SLOPE - TAPERED INLET



This inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends largely upon the amount of FALL available between the invert at the face and the invert at the throat section. Since this FALL may vary, a range of increased capacities is possible.

Slope-tapered inlets of alternate designs were considered and tested during the research. The inlet shown in Figure 4.6 is recommended on the basis of its hydraulic performance and ease of construction. As a result of the FALL concentrated between the face and the throat of this inlet, the barrel slope is flatter than the barrel slope of a conventional or side-tapered structure at the same site.

Both the face and throat are possible control sections in a slope-tapered inlet culvert. However, since the major cost of a culvert is in the barrel portion and not the inlet structure, the inlet face should be designed with a greater capacity at the allowable headwater elevation than the throat. This insures that flow control will be at the throat and more of the potential capacity of the barrel will be utilized.

Performance Curves

To understand how a culvert at a particular site will function over a range of discharges, a performance curve, which is a plot of discharge versus headwater depth or elevation, must be drawn. Figure 4.7 is a schematic performance curve for a culvert with either a side-tapered or slope-tapered

For these inlets, it is necessary to compute the performance of the face section (face control curve), the throat section (throat control curve), and the barrel (outlet control curve), in order to develop the culvert performance curve for a range of discharges. The actual culvert performance curve, the hatched line of Figure 4.7, represents the performance of the face, throat, and

barrel sections in the ranges where their individual performance determines the required headwater. In the lower discharge range, face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs.

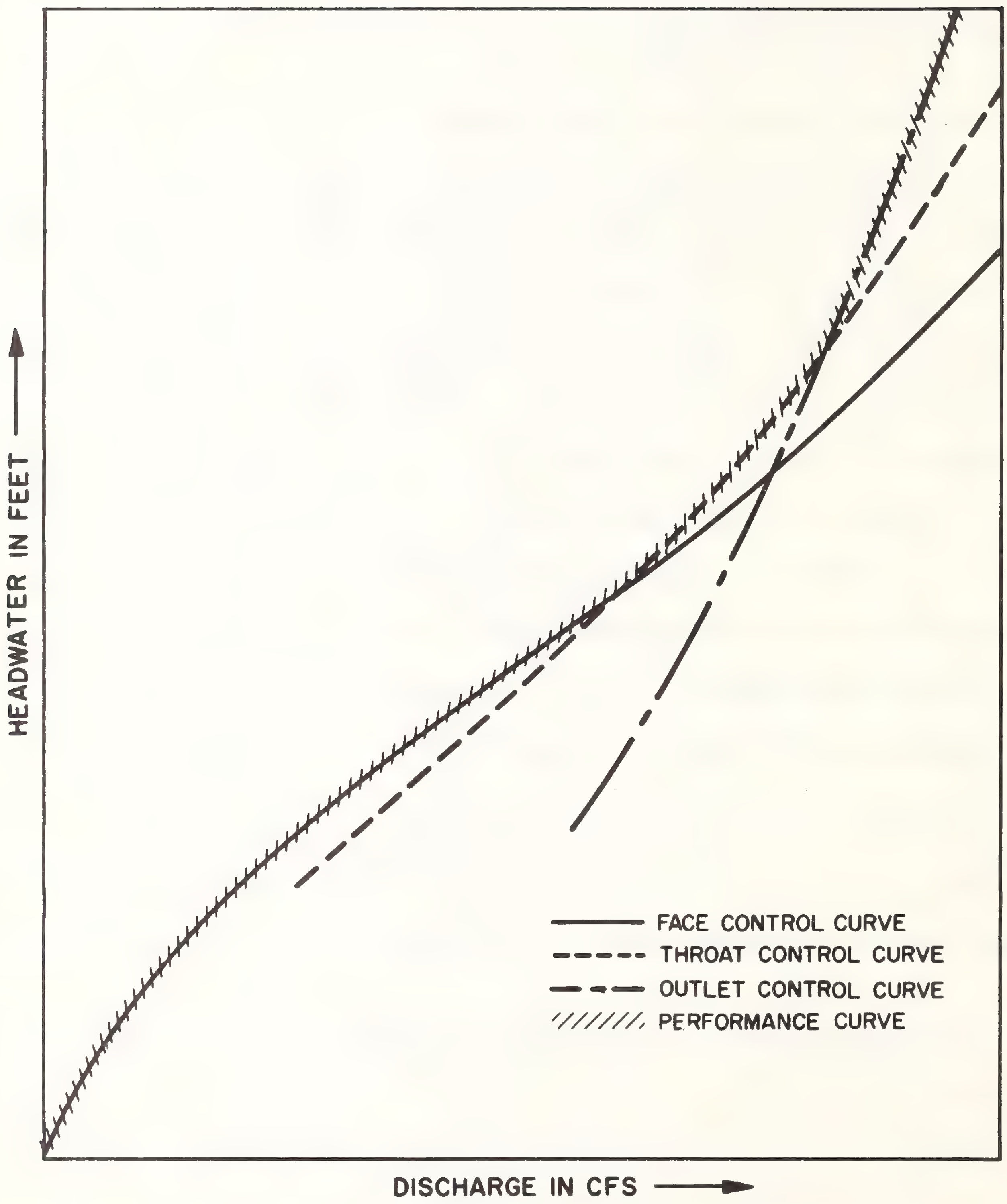
Performance curves should always be developed for culverts with side-tapered or slope-tapered inlets to insure that the designer is aware of how the culvert will function over a range of discharges, especially those exceeding the design discharge. It is important to emphasize that outlet control may govern for the larger discharges, and, as shown in Figure 4.7, the outlet control curve has a much steeper slope - a more rapidly rising headwater requirement for increasing discharges - than either the face or throat control curve. It should be recognized that there are uncertainties in the various methods of estimating flood peaks and that there is a chance that the design frequency flood will be exceeded during the life of the project. Culvert designs should be evaluated in terms of the potential for damage to the highway and adjacent property from floods greater than the design discharge.

As alternate culverts are possible using improved inlet design, a performance curve should be plotted for each alternate considered. The performance curve will provide a basis for selection of the most appropriate design.

The advantages of various improved inlet designs are demonstrated by the performance curves shown in Figure 4.8. These curves represent the performance of a single 6 foot by 6 foot reinforced concrete box culvert 200 feet long, with a 4 foot difference in elevation from the inlet to the outlet. For a given headwater, the culvert can convey a wide range of discharges, depending on the type of inlet used.

Curves 1 through 4 are inlet control curves for a 90 degree wingwall with a square-edged inlet, a 1.5:1 bevel-edged inlet, a side-tapered inlet, and a

Figure 4.7



SCHEMATIC PERFORMANCE CURVE

slope-tapered inlet with minimum FALL, respectively. Curves 5 and 6 are outlet control curves. Curve 5 is for the square-edged inlet and curve 6 is for the other three inlet types. As previously discussed, curves 5 and 6 show that improved entrances can increase the performance of a culvert operating in outlet control, but the improvment is not as great as for culverts operating in inlet control, as demonstrated by curves 1 through 4.

Tables A and B compare the inlet control performance of the different inlet types. Table A shows the increase in discharge that is possible for a headwater depth of 8 feet. The bevel-edged inlet, side-tapered inlet and slope-tapered inlet show increases in discharge over the square-edged inlet of 16.7, 30.4, and 55.6 percent, respectively. It should be noted that the slope-tapered inlet incorporates only the minimum FALL of D/4. Greater increases in capacity are often possible if a larger FALL is used.

TABLE A

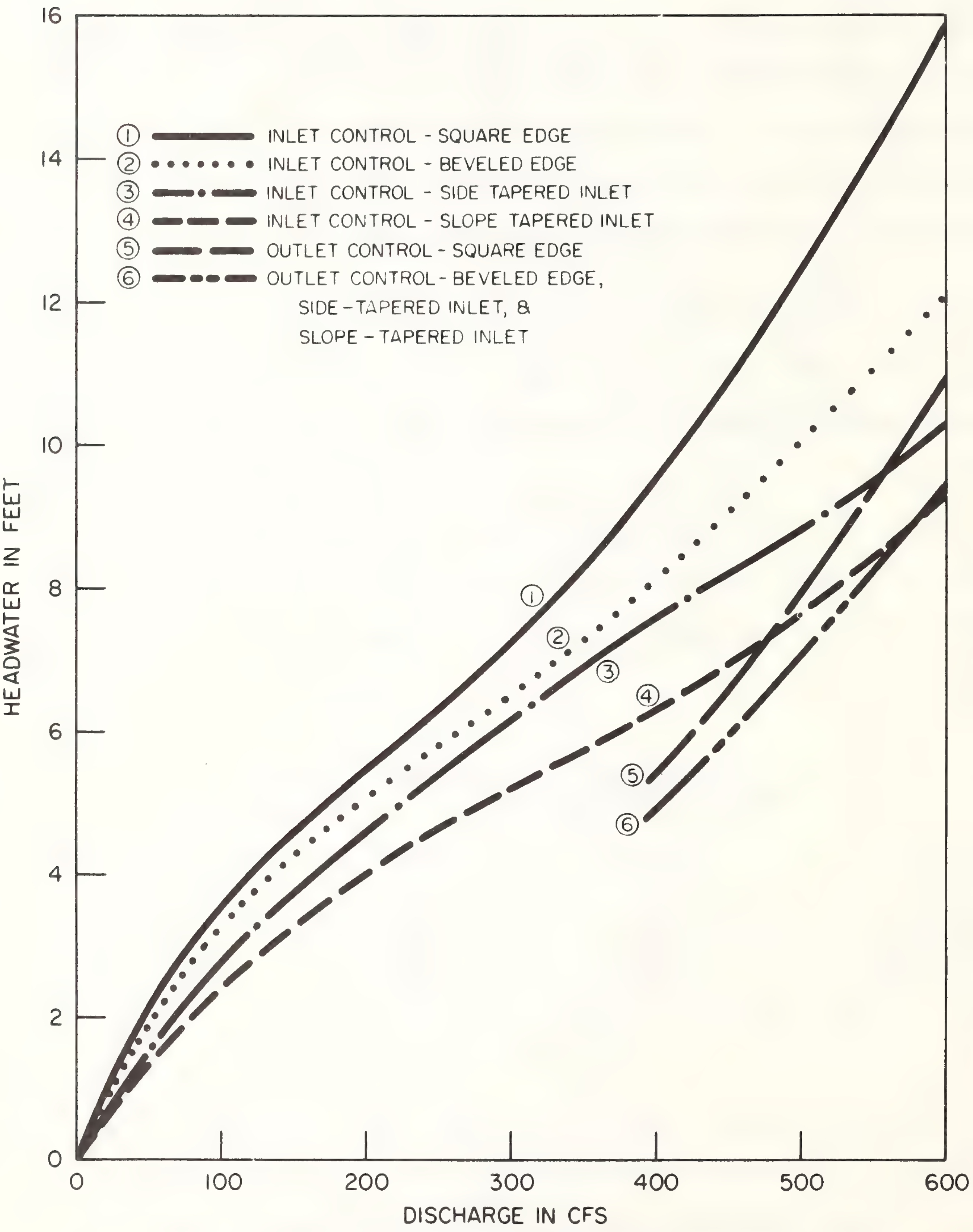
COMPARISON OF INLET PERFORMANCE AT
CONSTANT HEADWATER FOR 6 FT. X 6 FT. RCB

Inlet Type	Headwater	Discharge	% Improvement
Square-edge	8.0'	336 cfs	0
Bevel-edge	8.0'	392 cfs	16.7
Side-tapered	8.0'	438 cfs	30.4
*Slope-tapered	8.0'	523 cfs	55.6

*Minimum FALL in inlet = D/4 + 1.5 ft.

Table B depicts the reduction in headwater that is possible for a discharge of 500 cfs. The headwater varies from 12.5 ft. for the square-edged inlet to 7.6 ft. for the slope-tapered inlet. This is a 39.2 percent reduction in required headwater.

Figure 4.8



PERFORMANCE CURVES FOR
SINGLE 6' X 6' BOX CULVERT
90 DEGREE WINGWALL

TABLE B

COMPARISON OF INLET PERFORMANCE AT
CONSTANT HEADWATER FOR 6 FT. x 6 FT. RCB

Inlet Type	Headwater	Discharge	% Reduction
Square-edge	12.5'	500 cfs	0
Bevel-edge	10.1'	500 cfs	19.2
Side-Tapered	8.8'	500 cfs	29.6
*Slope-tapered	7.6'	500 cfs	39.2

*Mininum FALL in inlet = D/4 = 1.5 ft.

The performance curves in Figure 4.8 illustrate how inlet geometry affects the capacity of a given culvert. The practical use of performance curves to compare the operation of culverts of various sizes and entrance configurations for a given discharge are discussed in detail in later paragraphs.

In improved inlet design, the inverts of the face sections for the different types of improved inlets fall at various locations, depending on the design chosen. Therefore, it is difficult to define a datum point for use in comparing the performance of a series of improved inlet designs. The use of elevations is suggested, and this concept is used in the design procedure presented here. The example problem performance curves are plots of discharge versus required headwater elevations. Allowable headwater is also expressed as an elevation.

Box Culvert Improved Inlets

Bevel-Edged Inlets

Four inlet control charts for culverts with beveled edges are included in this section: Chart 4.16 for 90 degree headwalls (same as 90 degree wingwalls), Chart 4.17 for skewed headwalls, Chart 4.18 for wingwalls with flare angles of 18 to 45 degrees, and Chart 4.21 for circular pipe culverts with beveled rings. Note that Charts 4.16 through 4.18 apply only to bevels having either a 33.7 degree angle (1.5:1) or a 45 degree angle (1:1). For example, the minimum bevel dimension for an 8 ft. x 6 ft. box culvert designed using Chart 4.16 is a

1:1 bevel, or 45 degree angle, would be d = 6 ft. x 1/2 in./ft. = 3 in. and b = 8 ft. x 1/2 in./ft. = 4 in. Therefore, the top bevel would have a minimum height of 3 in., and the side bevel would be 4 in. in width. Similar computations would show that for a 1.5:1 or 33.7 degree angle, d would be 6 in. and b would be 8 in.

The design charts in this section are based on research results from culvert models with barrel width, B, to depth , D, ratio of from 0.5:1 to 2:1

Multibarrel Installations - For installations with more than one barrel, the nomographs are used in the same manner as for a single barrel, except that the bevels must be sized on the basis of the total clear opening rather than on individual barrel size. For example, in a double 8 ft. by 8 ft. box culvert, the top bevel is proportioned based on the height, 8 ft. and the side bevels proportioned based on the clear width, 16 feet. This results in a d dimension, for the top bevel of 4 in. for the 1:1 bevel, and 8 in. for the 1.5:1 bevel and ab dimension for the side bevels of 8 in. for the 1:1 bevel and 16 in. for the 1.5:1 bevel. The ratio of the inlet face area to the barrel area remains the same as for a single barrel culvert.

For multibarrel installations exceeding a 3:1 width to depth ratio, the side bevels become excessively large when proportioned on the basis of the total clear width. For these structures, it is recommended that the side bevel be sized in proportion to the total clear width, B, or three times the height, whichever is smaller. The top bevel dimension should always be based on the culvert height. Until further research information becomes available, the design charts may be used to estimate the hydraulic performance of these installations.

The shape of the upstream edge of the intermediate walls of multibarrel installations is not as important to the hydraulic performance of a culvert as

the edge conditions of the top and sides. Therefore, the edges of these walls may be square, rounded with a radius of one-half their thickness, chamfered, or beveled. The intermediate walls may also project from the face and slope downward to the channel bottom to act as debris fins as suggested in Section 4.15.

It is recommended that Chart 4.17 for skewed inlets not be used for multiple barrel installations, as the intermediate wall could cause an extreme contraction in the downstream barrels. This would result in underdesign due to a greatly reduced capacity. As discussed previously, skewed inlets should be avoided whenever possible, and should not be used with side- or slope-tapered inlets.

Side-Tapered Inlets

Description - The selected configurations of the side-tapered inlet are shown in Figure 4.9. The barrel and face heights are the same except for the addition of a top bevel at the face. Therefore, the enlarged area is obtained by making the face wider than the barrel and providing a tapered sidewall transition from the face to the barrel. Side taper ratios may range from 6:1 to 4:1. The 4:1 taper is recommended as it results in a shorter inlet.

The throat and the face are possible flow control sections in the side-tapered inlet. The weir crest is a third possible control section when a FALL is used. Each of the possible control sections should be sized to pass the design discharge without exceeding the allowable headwater elevation. Plots of the performance of each of the possible inlet control sections along with the outlet control performance curve define the culvert performance.

Throat Control - In order to utilize more of the available culvert barrel area, the control at design discharge generally should be at the throat rather than at the face or crest. Chart 4.26 presents the headwater depth,

referenced to the throat invert, required to pass a given discharge for side- and slope-tapered inlets operating in throat control. This chart is in a semi-dimensionless form, H_t/D plotted against $Q/BD^{3/2}$. The term, $Q/BD^{3/2}$, is not truly dimensionless, but is a convenient parameter and can be made non-dimensional by dividing by the square root of gravitational acceleration, $g^{1/2}$.

Face Control - Design curves for determining face width are provided in Chart 4.27. Both the inlet edge condition and sidewall flare angle affect the performance of the face section. The two curves in Chart 4.27 pertain to the options in Figure 4.11. The dashed curve, which is less favorable, applies to the following inlet edge conditions:

(1) wingwall flares of 15 degrees to 26 degrees and a 1:1 top edge bevel;

(2) wingwall flares of 26 degrees thru 90 degrees and square edges (no bevels). A 90 degree wingwall flare is commonly termed a headwall.

The more desirable solid curve applies to the following entrance conditions:

(1) wingwall flares of 26 degrees to 45 degrees with a 1:1 top edge bevel, or

(2) wingwall flares of 45 degrees to 90 degrees with a 1:1 bevel on the side and top edges.

Note that undesirable design features, such as wingwall flare angles less than 15 degrees, or 26 degrees without a top bevel, are not covered by the charts. Although the 1.5:1 bevels can be used, due to structural consideration, the smaller 1:1 bevels are preferred.

Use of FALL Upstream of Side-Tapered Inlet - A depression may be utilized upstream of the face of a side-tapered inlet. As illustrated in Figures 4.9 and 4.10, the depression may be constructed in various ways, as an extension of the wingwalls, or by a paved depression similar to that used with side-tapered pipe culvert inlets, shown in Figure 4.16. The only requirements are:

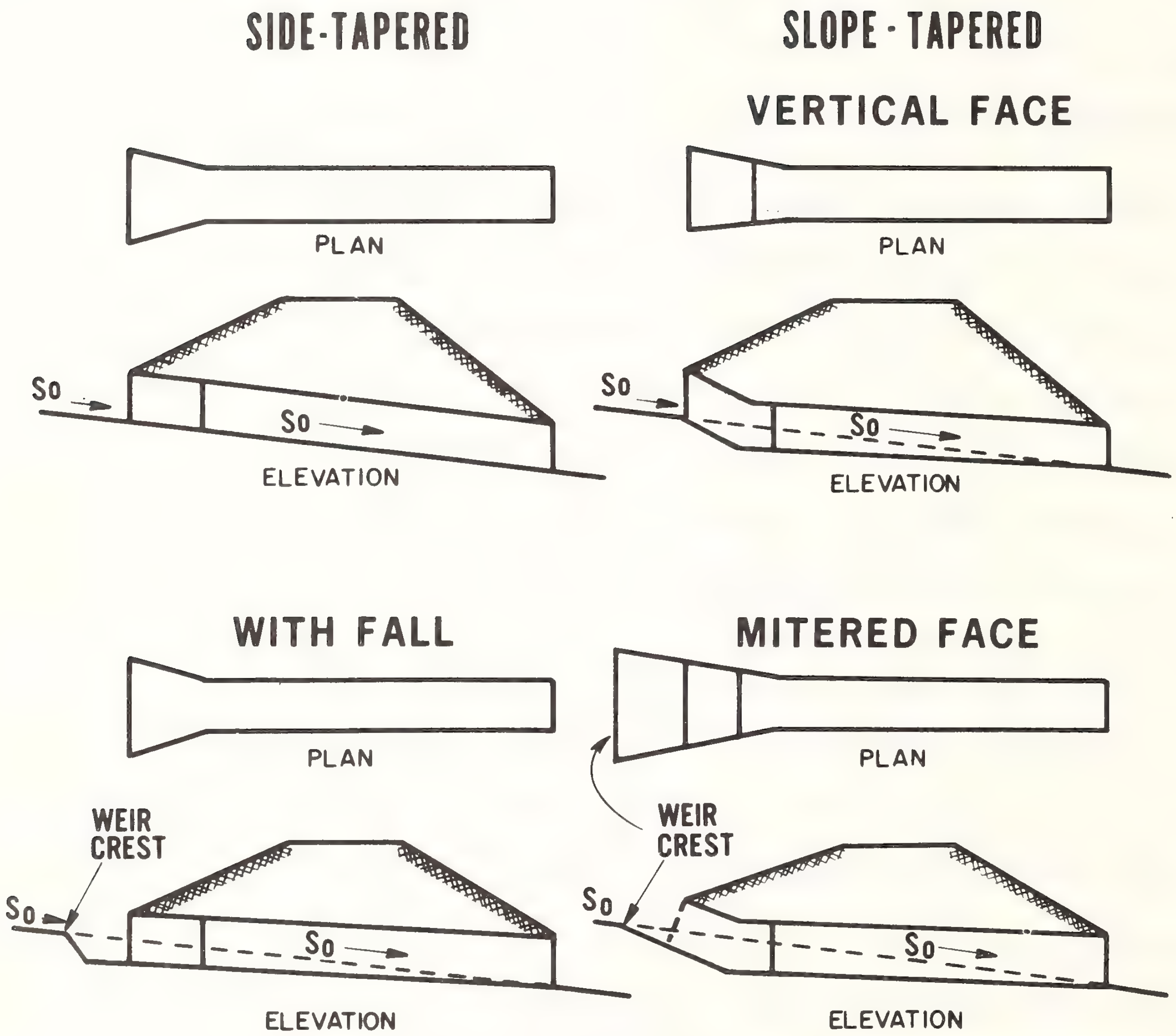
the plane of the invert of the barrel be extended upstream from the inlet face a minimum distance of D/2, to provide a smooth flow transition into the inlet; and, the crest be long enough to avoid undesirably high headwater from crest control at design discharges. Chart 4.29 may be used for checking crest control if the fall slope is between 2:1 to 3:1. The length of the crest, W, may be approximated, neglecting flow over the sides of sloping wingwalls. This provides a conservative answer.

Performance Curves - Figure 4.12 illustrates the design use of performance curves and shows how the side-tapered inlet can reduce the barrel size required for a given discharge. (The detailed calculations for Figure 4.12 are given in Example Problem No. 1). Performance curve No. 1 is for a double 7 ft. x 6 ft. conventional culvert with 90 degree wingwalls (headwall) and 1:1 bevels on both the top and side. This conventional inlet will be the "standard" to which curves for the improved inlets may be compared.

The hatched performance curve is for a double 6 ft. x 5 ft. box culvert with a side-tapered inlet with no FALL upstream. It is a composite of the throat and face control curves. The outlet control curve was also computed, but falls outside of the limits of the figure. This indicates that further increases in capacity or reduction in headwater are possible. Face control governs to a discharge of 375 cfs, and throat control for larger discharges. Thus, the barrel dimensions (throat size) control the designs at high discharges, which should always be the case. In this example, the size of the culvert was reduced from a double 7 ft. x 6 ft. box to a double 6 ft. x 5 ft. for the same allowable headwater. Use of an upstream FALL would reduce the barrel size still further to a size comparable to that required with a slope-tapered inlet.

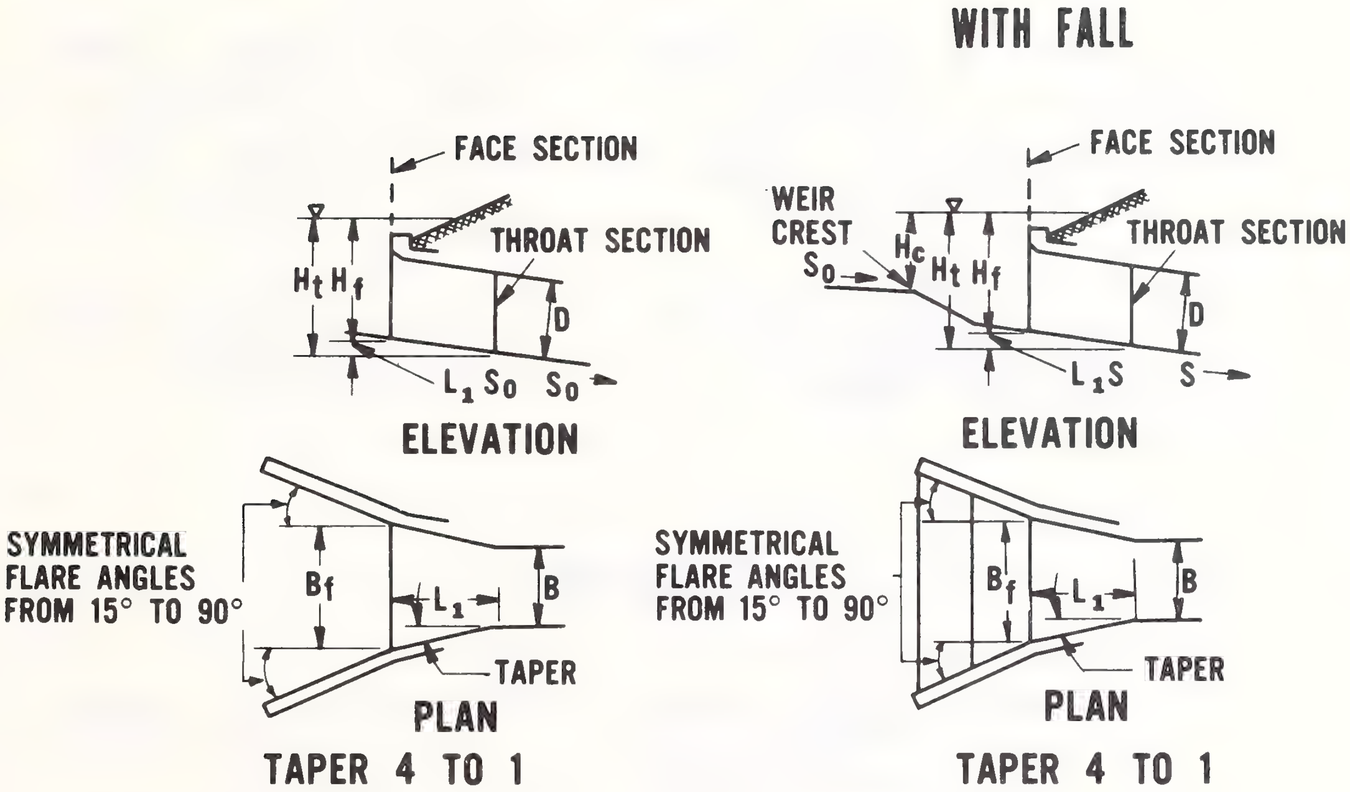
Double Barrel Design - As shown in the above example, double barrel structures may be designed with improved inlets. The face is proportioned

Figure 4.9



TYPES OF IMPROVED INLETS FOR BOX CULVERTS

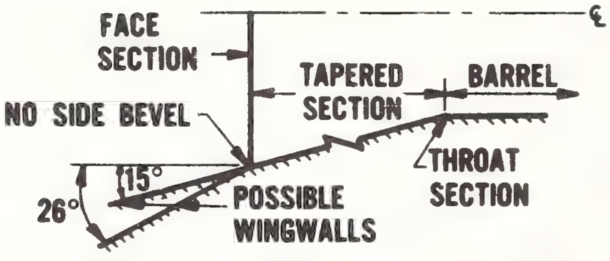
Figure 4.10



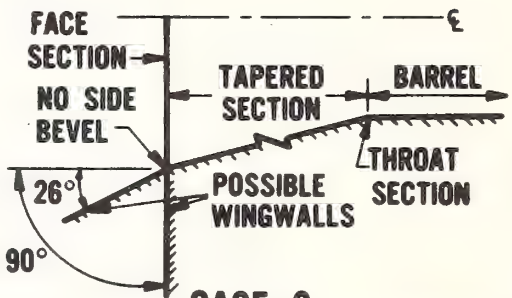
IMPROVED INLETS SIDE-TAPERED

Figure 4.11

DASHED CURVE

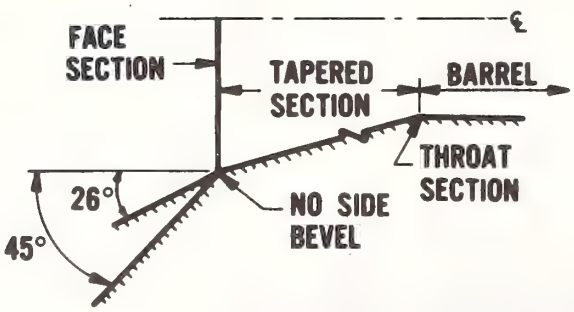


CASE 1
15° TO 26° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED

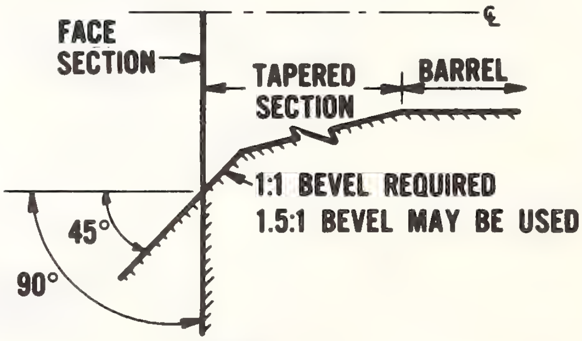


CASE 2
26° TO 90° WINGWALL FLARE ANGLES
NO TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED

SOLID CURVE



CASE 1
26° TO 45° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
NO SIDE BEVEL REQUIRED



CASE 2
45° TO 90° WINGWALL FLARE ANGLES
TOP BEVEL REQUIRED
SIDE BEVEL REQUIRED

DEFINITION OF CURVES ON FACE CONTROL DESIGN CHARTS 4.27 & 4.28

on the basis of the total clear width as described for bevels. The center wall is extended to the face section with either a square, rounded, chamfered, or beveled edge treatment. A sidewall taper of from 4:1 to 6:1 may be used.

The face width, as determined from Chart 4.27, is the total clear face width needed. The width of the center wall must be added to this value in order to size the face correctly.

No design procedure is available for side-tapered inlet culverts with more than two barrels.

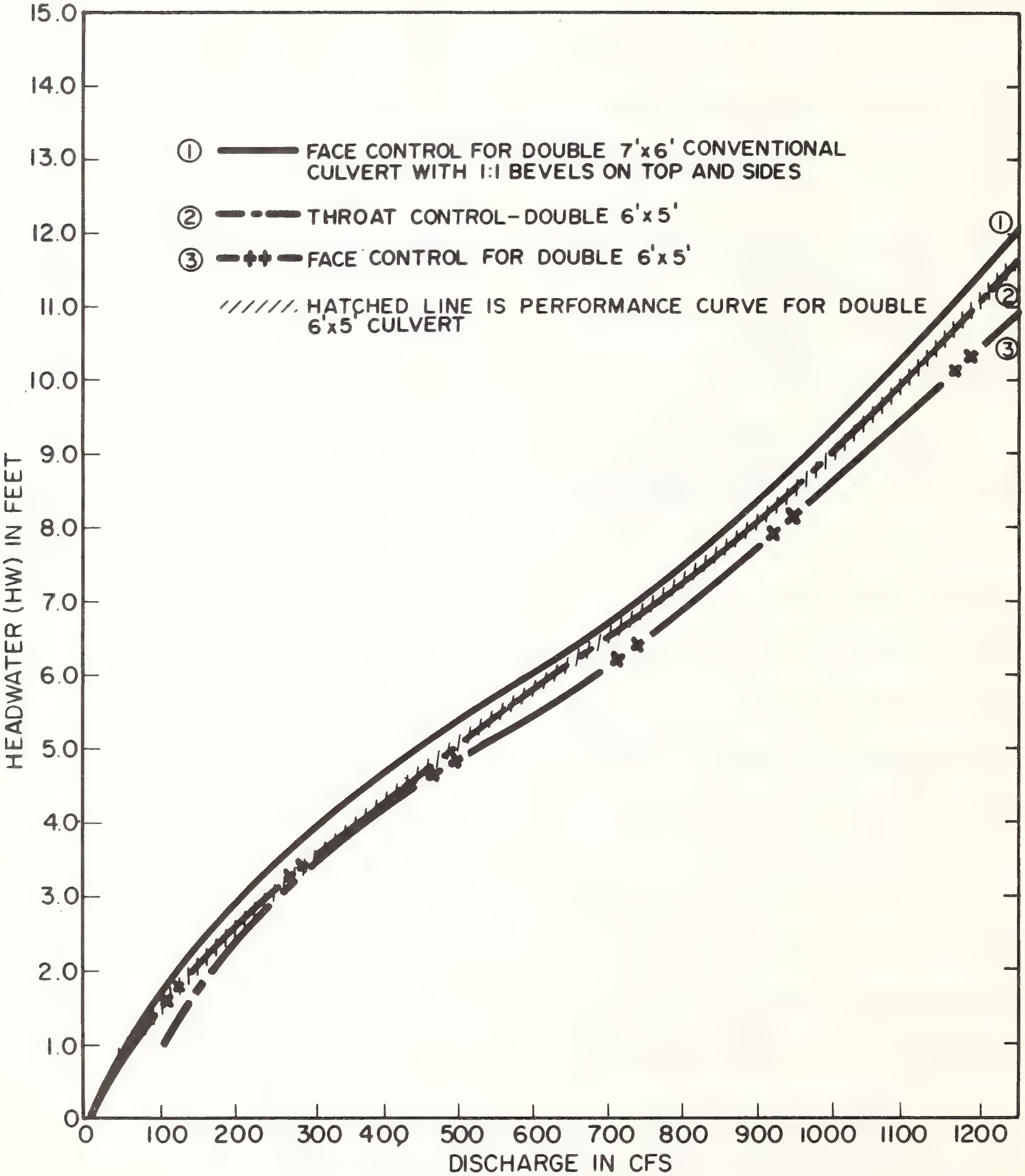
Slope-Tapered Inlets

The inlets shown in Figure 4.13 are variations of the slope-tapered inlet and provide additional improvements in hydraulic performance by increasing the head on the control section. The difference between the two types of slope-tapered inlets lies in the face section placements. One type has a vertical face configuration and the other a mitered face. The face capacity of the latter type is not based on its physical face section, but on a section perpendicular to the fall slope intersecting the upper edge of the opening. This is illustrated by the dashed line in Figure 4.13.

Excluding outlet control operation, the slope-tapered inlet with a vertical face has three potential control sections: the face, the throat, and the bend (Figure 4.13). The bend is located at the intersection of the fall slope and the barrel slope. The distance, L_3, between the bend and the throat must be at least 0.5B, measured at the soffit or top of the culvert, to assure that the bend section will not control. Therefore, the hydraulic performance needs only be evaluated at the face and throat sections. The slope-tapered inlet with a mitered face has a fourth possible control section, the weir crest.

Throat Control - As with side-tapered inlets, throat control performance should usually govern in design since the major cost is in the construction of

Figure 4.12



PERFORMANCE CURVES FOR DIFFERENT BOX CULVERTS WITH VARYING INLET CONDITIONS

(SIDE-TAPERED INLET)

the barrel. Chart 4.26 is the throat control design curve for both slope-tapered inlets. By entering Chart 4.26 with a computed value for $Q/BD^{3/2}$, H_t can be determined from the value $\frac{H_t}{D}$.

Face Control - Face control design curves for slope-tapered inlets are presented in Chart 4.28. The two design curves apply to the face edge and wingwall conditions shown in Figure 4.11.

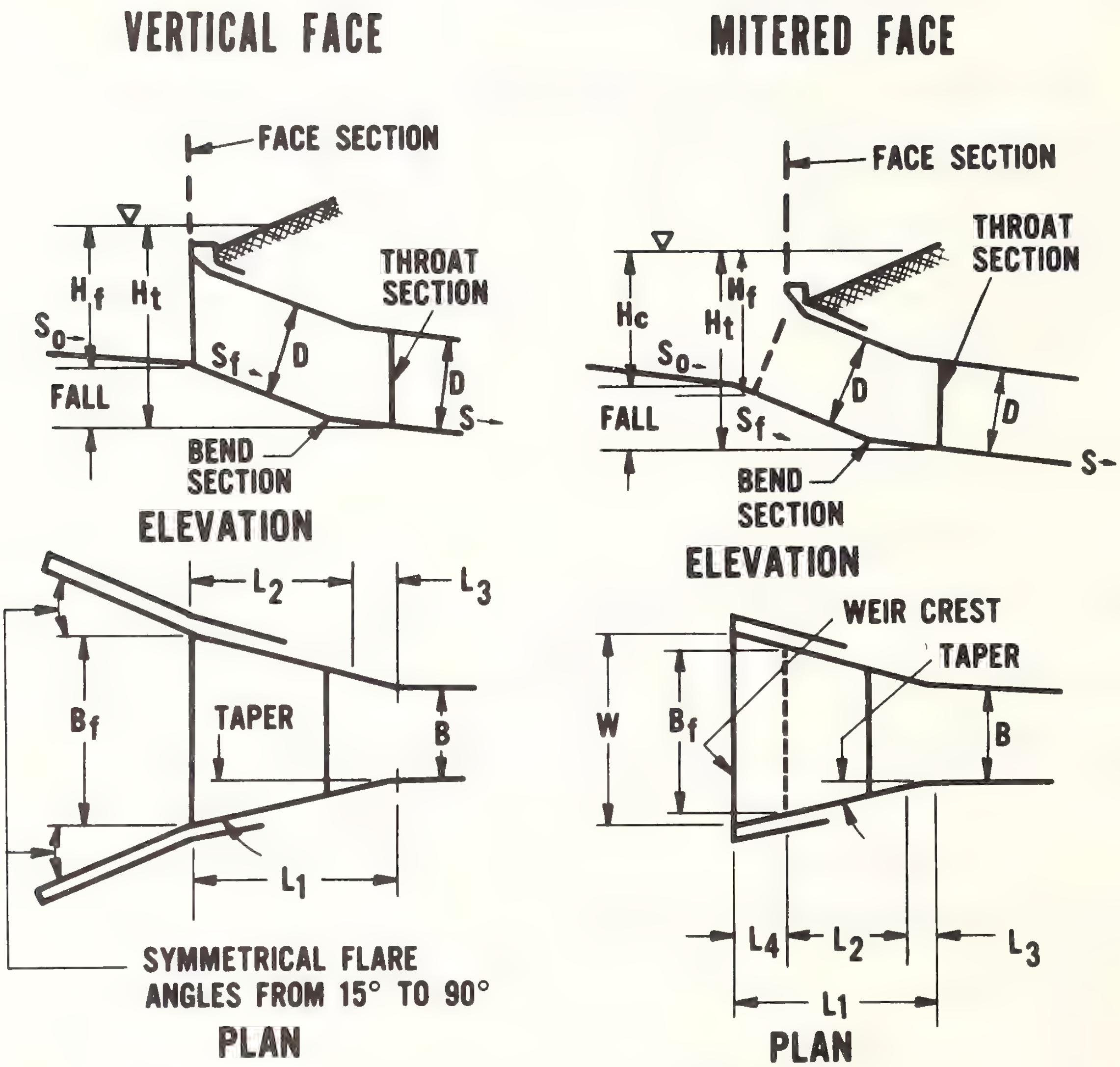
Crest Control - The possibility of crest control should be examined for the slope-tapered inlet with a mitered face using Chart 4.29. The crest width, W, is shown in Figure 4.13. Again, there may be flow from the sides over the wingwalls, but generally this can be neglected. As the headwater rises above the wingwalls, there is little chance that the crest will remain the control section.

Design Limitations - In the design of slope-tapered inlets, the following limitations are necessary to insure that the design curves provided will always be applicable. If these limitations are not met, hydraulic performance will not be as predicted by design curves given in this section.

> The <u>fall slope</u> must range from 2:1 to 3:1. Fall slopes steeper than 2:1 have adverse performance characteristics and the design curves do not apply. If a fall slope less than 3:1 is used, revert to design Chart 4.27 for side-tapered inlets and use the fall slope that is available. <u>Do not</u> interpolate between Charts 4.27 and 4.28.

> The FALL should range from D/4 to 1.5D for direct use of the curves. For FALLS greater than 1.5D, frictional losses between the face and the throat must be calculated and added to the headwater. For FALLS less than D/4, use design Chart 4.27 for side-tapered inlets and the FALL that is available. <u>Do not</u> interpolate between Charts 4.27 and 4.28.

Figure 4.13



IMPROVED INLETS
SLOPE-TAPERED

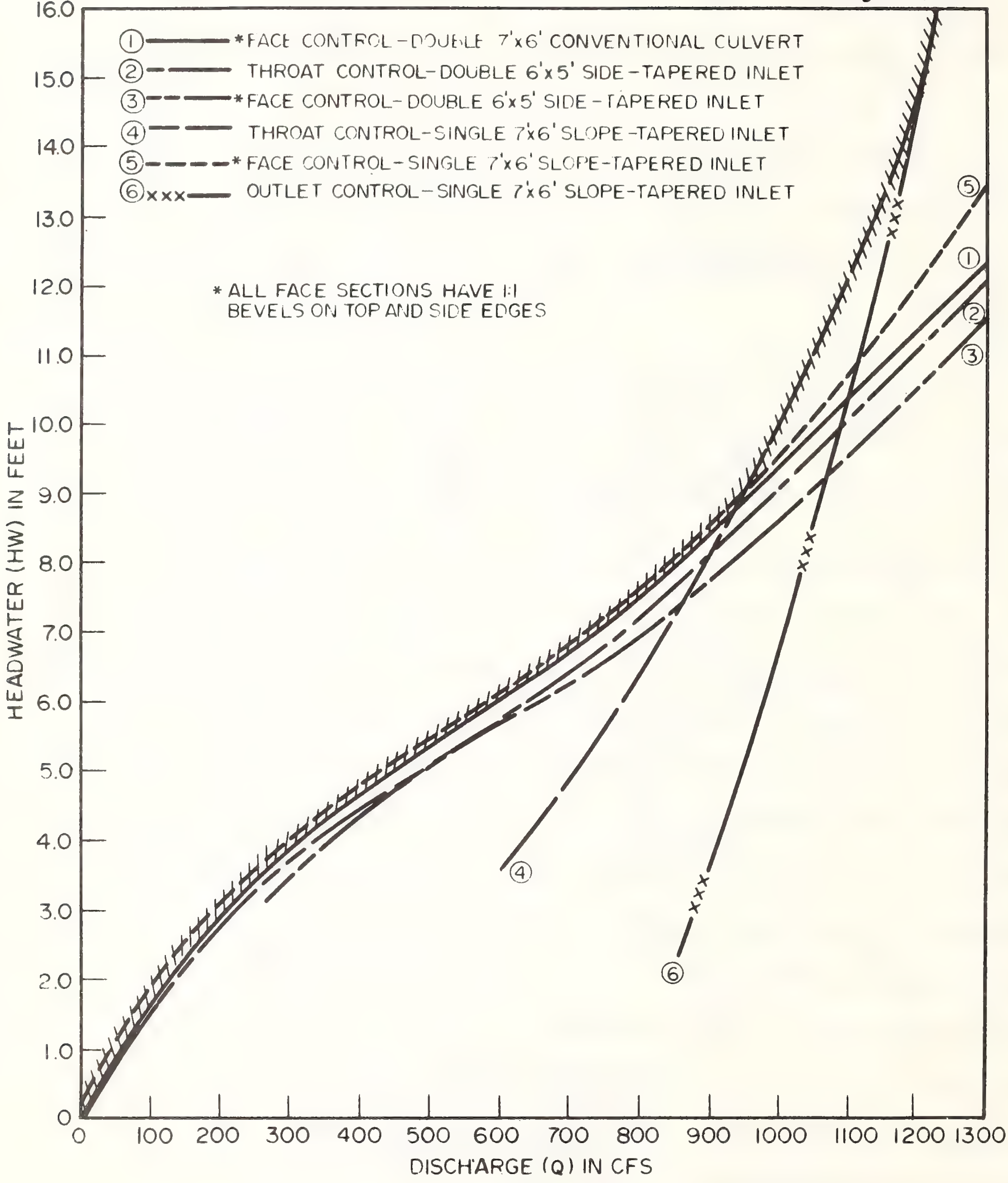
The sidewall taper should be from 4:1 to 6:1. Tapers less than 4:1 are unacceptable. Tapers greater than 6:1 will perform better than the design curves indicate, and the design will be conservative.

L_3 must be a minimum of 0.5B measured at the soffit or inside top of the culvert. Larger values may be used, but smaller ones will cause the area provided for the bend to be so reduced that the bend section will control rather than the throat section. Do not use an L_3 value less than 0.5B.

Performance Curve - In Figure 4.14, performance curves for the slope-tapered inlet are shown in addition to the performance curves shown in Figure 4.12. Detailed calculations may be found in Example 1.

As can be seen from Figure 4.14, the performance of a single 7 ft. by 6 ft. culvert with a slope-tapered inlet is comparable to a double conventional 7 ft. by 6 ft. culvert with beveled edges. Note that the performance curve for the single 7 ft. by 6 ft. culvert (hatched line) is developed from the face control curve (Curve 5) from 0 to 950 cfs, the throat control curve (Curve 4) from 950 to 1,200 cfs and the outlet control curve (Curve 6) for all discharges above 1,200 cfs. This illustrates the need for computing and plotting the performance of each control section and demonstrates the barrel size reduction possible through use of improved inlets. The performance curves clearly indicate the headwater elevation requeired to pass any discharge. This is an invaluable tool in assessing the consequences of a flood occurrence exceeding the design discharge estimate. The use of performance curves in maximizing performance and optimization of design will be discussed later.

Double Barrel Design - Charts 4.26, 4.28, and 4.29 depict single barrel installations, but they are applicable to double barrel installations with the center wall extended to the face section.



PERFORMANCE CURVES FOR DIFFERENT
BOX CULVERTS WITH VARYING INLET CONDITIONS

In addition to the comments and limitations for single barrel slope-tapered inlets, the face must be proportioned on the basis of the total clear width. The center wall is extended to the face section and may have any desired edge treatment.

The face width, as determined from Chart 4.28, is the total clear face width. The center wall width must be added to the value found from Chart 4.28 in order to size the face correctly.

No design procedure is available for slope-tapered inlet culverts with more than two barrels.

Pipe Culvert Improved Inlets

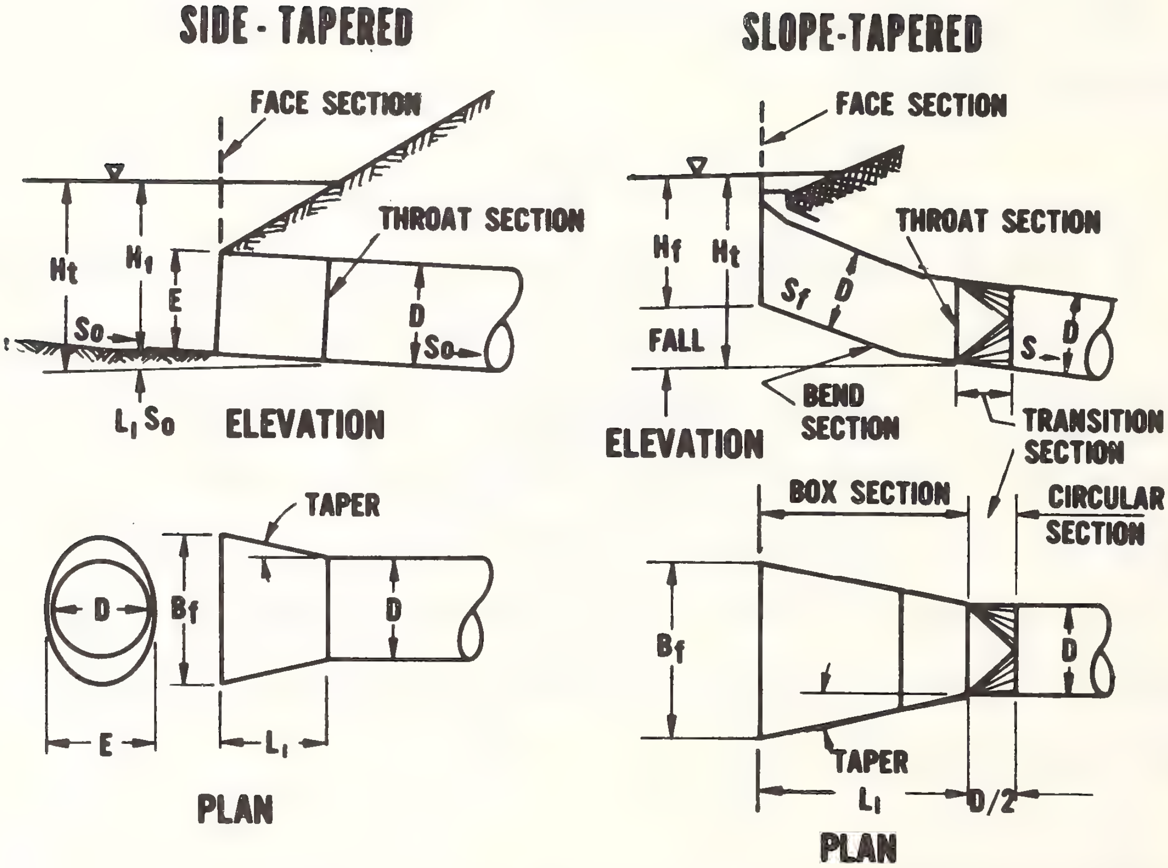
As with box culverts, for each degree of pipe culvert inlet improvement there are many possible variations using bevels, tapers, drops, and combinations of the three. The tapered inlets are generally classified, as shown in Figure 4.15, as either side-tapered (flared) or slope-tapered. The side-tapered inlet for pipe culverts is designed in a manner similar to that used for a side-tapered box culvert inlet. The slope-tapered design for pipes utilizes a rectangular inlet with a transition section between the square and round throat sections.

Beveled-Edged Inlets

Design charts for conventional pipe culverts with different entrances edge conditions are contained in this section. As previously mentioned, the socket end of a concrete pipe results in about the same degree of hydrualic improvement as a beveled edge. Therefore, it is suggested that the socket be retained at the upstream end of concrete pipes, even if some warping of the fill slope is required because of the longer pipe or skewed installation.

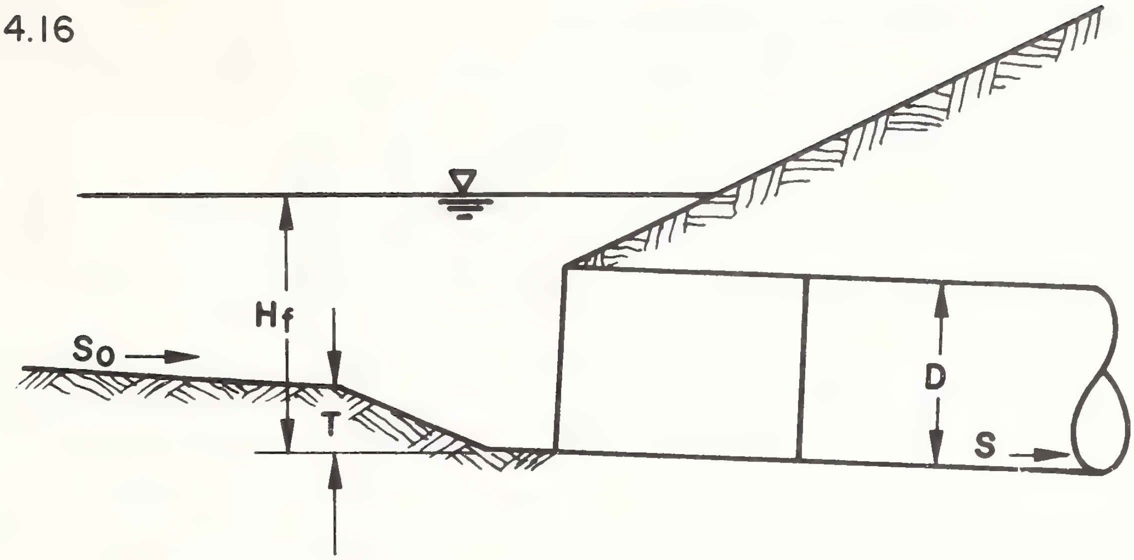
Multibarrel pipe culverts should be designed as a series of single barrel

Figure 4.15

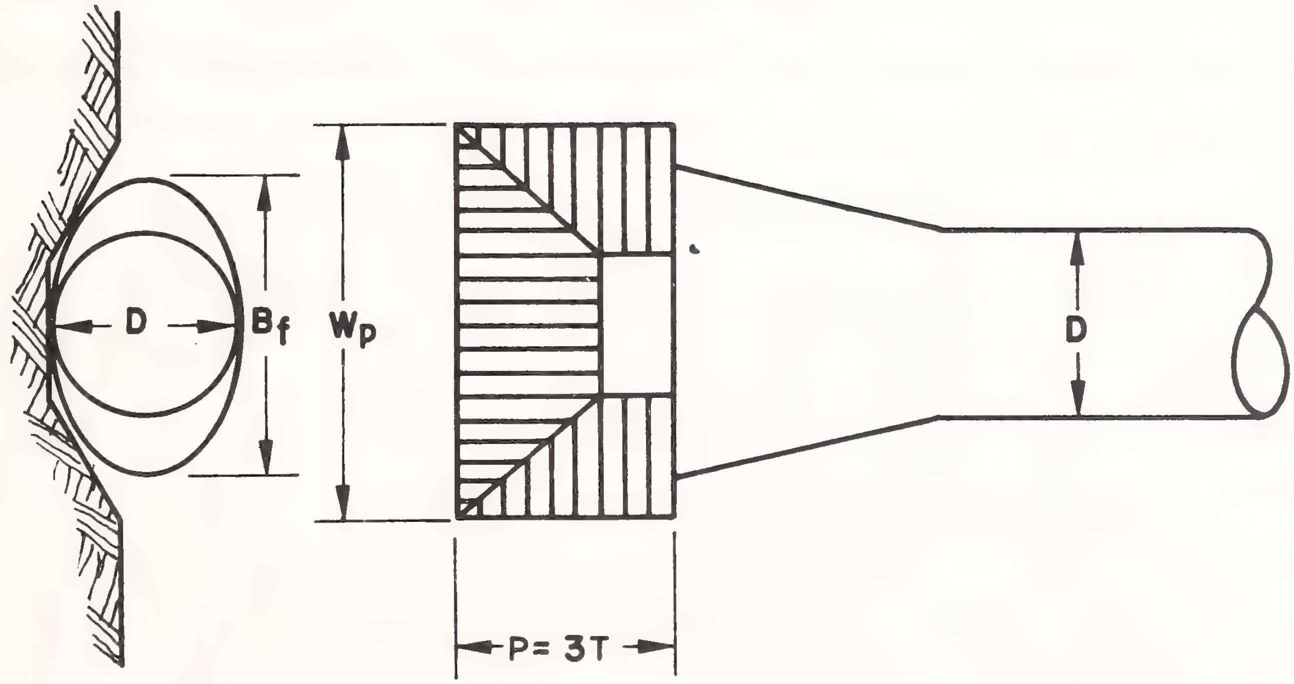


TYPES OF IMPROVED INLETS FOR PIPE CULVERTS

Figure 4.16



ELEVATION



PLAN

W_p = Bf+T or 4T WHICH EVER IS LARGER

SIDE-TAPERED INLET WITH CHANNEL DEPRESSION UPSTREAM OF ENTRANCE

installation using the appropriate design charts since each pipe requires a separate bevel.

Side-Tapered Pipe Inlets (Flared Inlets)

Description - The side-tapered or flared inlet shown in Figure 4.15 is comparable to the side-tapered box culvert inlet. The face area is larger than the barrel area and may be in the shape of an oval, as shown in Figure 4.15, a circle, a circular segment, or a pipe-arch. The only limitations on face shape are that the vertical face dimension, E, be equal to or greater than D and equal to or less than 1.1D and that only the above face shapes be used with inlets designed using Chart 4.31. Retangular faces may be used in a manner similar to that described for the side- and slope-tapered inlet. The side taper should range from 4:1 to 6:1.

As with the box culvert side-tapered inlet, there are two possible control sections: the face and the throat (Figure 4.15). In addition, if a depression is placed in front of the face, the crest may control. The variation of the side-tapered inlet is depicted in Figure 4.16, and will be discussed later.

Throat Control - As stated before, the barrel of a culvert is the item of greatest cost; therefore, throat control should govern in the design of all improved inlets. Throat control design curves for side-tapered inlets are presented in Chart 4.30. Note that this chart contains two throat control design curves while the box culvert charts have only one. One curve is for entrances termed "smooth", such as those built of concrete or smooth metal, and the other is for "rough" inlets, such as those built of corrugated metal. The need for two curves results from different roughness characteristics and the difference in energy losses due to friction between the face and throat of the inlets.

Chart 4.30 applies only to circular barrels. It should not be used for rectangular, pipe-arch, or oval sections. Chart 4.26 is used for rectangular

sections, but no information is available for using improved inlets with pipe-arch or oval barrels.

Face Control - Face control for the side-tapered pipe culvert inlet are presented in Chart 4.31. The three curves on this chart are for: the thin-edged projecting inlet, the square-edged inlet, and the bevel-edged inlet. Note that the headwater is given as a ratio of E rather than D. This permits the use of the curves for face heights from D to 1.1D, as the equations used in developing the curves do not vary within this range of E.

In Chart 4.31, flexibility is allowed in choosing the face shape by presenting the flow rate, A, in terms of $Q/A_fE^{1/2}$, rather than $D^{5/2}$. By using the area of the face, A_f, and its height, E, the designer may choose or evaluate any available shape, such as elliptical, circular, a circular segment, or a pipe-arch. However, this chart does not apply to rectangular face shapes.

Fall Upstream of Inlet Face - In order to provide additional head for the throat section of pipe culverts, the slope-tapered inlet may be used, or a depression can be placed upstream of the side-tapered inlet face. There are various methods of constructing such a depression, including a drop similar to that shown for the side-tapered box of a constantly sloping bottom from the crest to a point a minimum distance of D/2 upstream of the face invert, and on line with the barrel invert. Chart 4.29 should be used to assure that the weir crest is long enough to avoid crest control.

Another means of providing a FALL upstream of the face is depicted in Figure 4.16. This configuration can be used with 90 degree wingwall (headwall). The depression will probably require paving to control upstream erosion. Research results indicated that such a depression could cause a moderate decrease in the performance of the face. To insure that this reduction in performance is not extreme, the following dimensional considerations should

be observed (Figure 4.16):

1. The minimum length of the depression, R, should be 3T;
2. The minimum width, WP, of the depression should be B_f + T or 4T, whichever is larger;
3. The crest length should be taken as W_p + 2(P) when using 17 to determine the minimum required weir length.

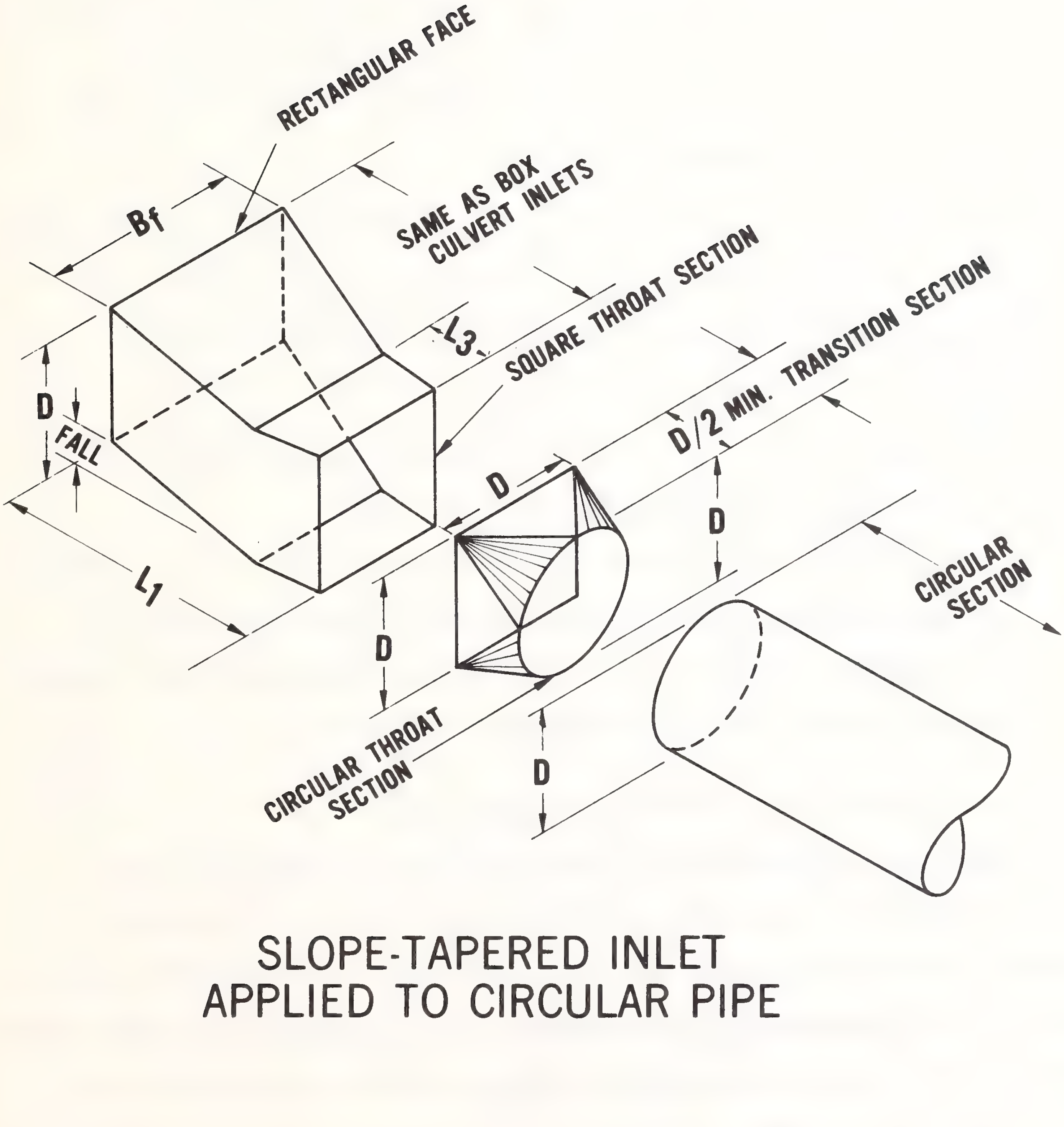
<u>Slope-Tapered Inlets for Pipe Culverts</u>

In order to utilize more of the available total culvert fall in the inlet area, as is possible with the box culvert slope-tapered inlets, a method was devised to adapt rectangular inlets to pipe culverts as shown in Figure 4.17. As noted in the sketch, the slope-tapered inlet is connected to the pipe culvert by use of a square to circular transition over a minimum length of one-half the pipe diameter. The design of this inlet is the same as presented in the box culvert section. There are two throat sections, one square and one circular, and the circular throat section must be checked by use of Chart 4.30. In all cases, the circular throat will govern the design because its area is much smaller than the square throat section. Thus, the square throat section need not be checked. The culvert performance curve consists of a composite of performance curves for the inlet control sections and the outlet control performance curve.

Square to round transition sections have been widely used in water resource projects. They are commonly built in-place, but also have been preformed. It is recommended that plans permit prefabrication or precasting as an alternate to in-place construction.

<u>Rectangular Side-Tapered Inlets for Pipe Culverts</u>

The expedient suggested for adapting the slope-tapered inlet for use with pipe culverts can also be used on side-tapered inlets where unusually large pipes



SLOPE-TAPERED INLET
APPLIED TO CIRCULAR PIPE

or sizes not commonly used are encountered. It may not be economical to prefabricate or precast a "one-of-a-kind" side-tapered or flared inlet, in which case, a cast-in-place rectangular side-tapered inlet would be a logical bid alternate. Also, flared inlets for large pipes may be too large to transport or to handle on the job. In this case, the flared or side-tapered pipe inlet could either be prefabricated or precast in two sections or the rectangular side-tapered inlet may be used as a bid or design alternate. Information for determining throat and face control performance is provided in Charts 4.30 and 4.27, respectively.

Design Limitations - In addition to the design limitations given previously for box culvert slope-tapered inlets, the following criteria apply to pipe culvert slope-tapered inlets and rectangular side-tapered inlets for pipe culverts:

1. The rectangular throat of the inlet must be a square section with sides equal to the diameter of the pipe culvert.
2. The transition from the square throat section to the circular throat section must be no shorter than one-half the culvert diameter D/2. If excessive lengths are used, the frictional loss within this section of the culvert should be considered in the design.

Multibarrel Designs - The design of multiple barrels for circular culverts using slope-tapered improved inlets can be performed the same as for box culverts, except that the center wall must be flared in order to provide adequate space between the pipes for proper compaction of the backfill. The amount of flare required will depend on the size of the pipes and the construction techniques used. No more than two barrels may feed from the inlet structure using these design methods.

An alternative would be to design a series of individual circular culverts

with slope-tapered inlets. This permits the use of an unlimited number of barrels, and the curves and charts of this publication are applicable.

Oval and Arch Pipe Improved Inlet Design

The hydraulic performance of oval pipes is given in Chart No. 4.5 for outlet control and Charts No. 4.22 and 4.23 for inlet control. The hydraulic performance of arch pipes is given in Charts No. 4.6, 4.7, and 4.8 for outlet control and Charts No. 4.24 and 4.25 for inlet control.

Coefficients and curves for the design of improved inlets for oval and arch pipes have not been developed yet. However, the improved inlet concepts do apply to these shapes. The general concepts should be applied to the design of any inlet structure that is to be placed on arch or oval culverts. When the coefficients and curves necessary for the improved inlet design are developed they will be included in this manual.

Design Procedure

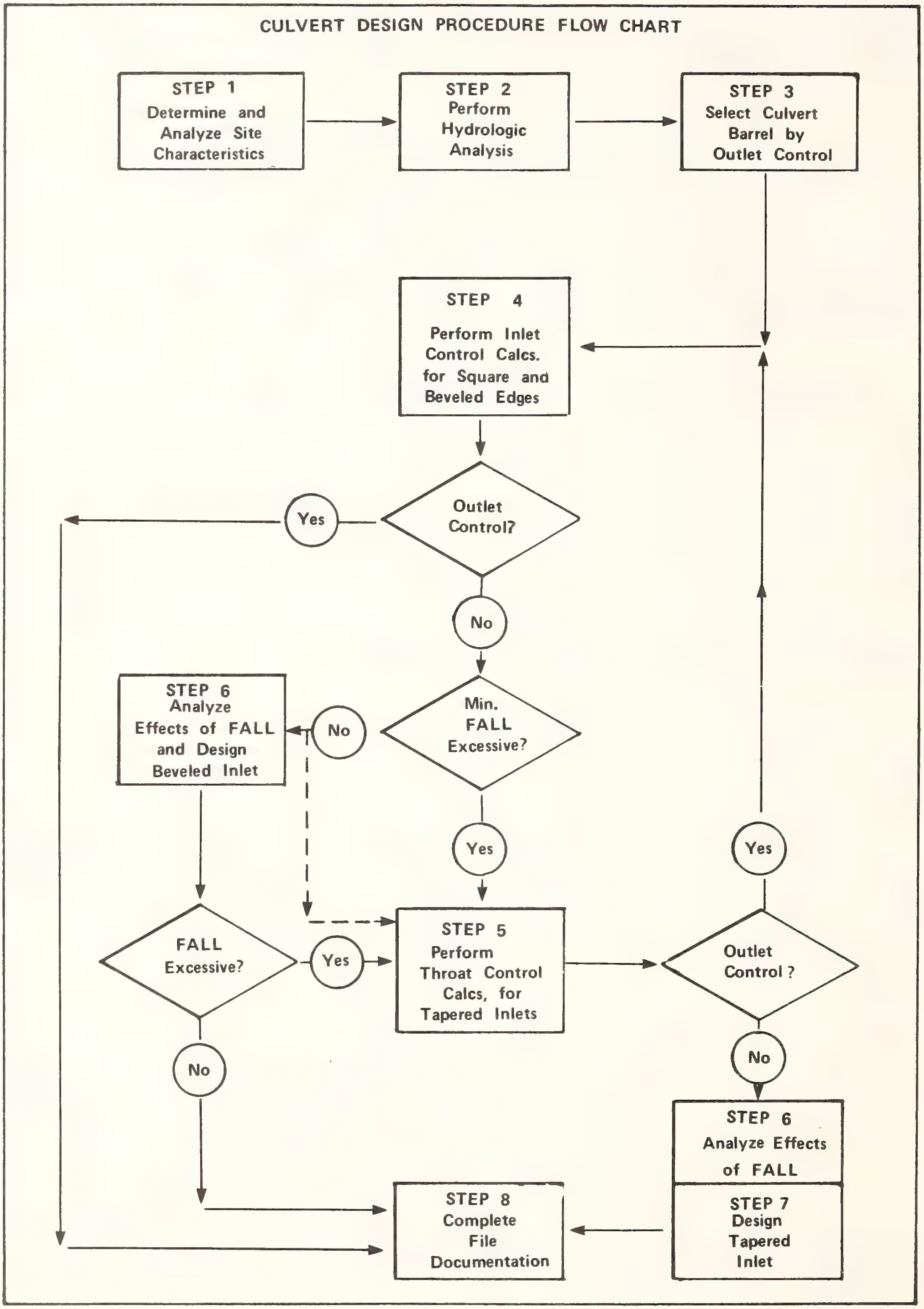
General

The objective of this section is to present the procedure of the hydraulic design of culverts, using improved inlets where appropriate. Economic considerations, although not specifically discussed, are implied in the design procedure.

The design procedure hinges on the selection of a culvert barrel based on its outlet control performance curve, which is unique when based on elevation. The culvert inlet is then manipulated using edge improvements and adjustments of its control performance. The resultant culvert design will best satisfy the criteria set by the designer and make optimum use of the barrel selected for the site.

The flow chart shown in Figure 4.18 outlines the steps of the design procedure, and each step is discussed in detail below. All calculations

Figure 4.18


CULVERT DESIGN PROCEDURE FLOW CHART
STEP 1 Determine and Analyze Site Characteristics
STEP 2 Perform Hydrologic Analysis
STEP 3 Select Culvert Barrel by Outlet Control
STEP 4 Perform Inlet Control Calcs. for Square and Beveled Edges
Outlet Control?
Yes
No
Min. FALL Excessive?
No
Yes
STEP 6 Analyze Effects of FALL and Design Beveled Inlet
FALL Excessive?
Yes
No
STEP 5 Perform Throat Control Calcs, for Tapered Inlets
Outlet Control ?
Yes
No
STEP 6 Analyze Effects of FALL
STEP 7 Design Tapered Inlet
STEP 8 Complete File Documentation

should be shown on the culvert design calculation sheets.

Step 1. Determine and Analyze Site Characteristics

Site characteristics include the generalized shape of the highway embankment, bottom elevations and cross sections along the stream bed, the approximate length of the culvert, and the allowable headwater elevation. In determining the allowable headwater elevation (AHW El.), roadway elevations and the elevation of upstream property should be evaluated and kept in mind throughout the design process. In some instances, such as in unpopulated rural areas, little or no damage would result, while at some sites great losses may ensue.

Culvert design is actually a trial-and-error procedure because the length of the barrel cannot be accurately determined until the size is shown, and the size cannot be precisely determined until the length is known. In most cases, however, a reasonable estimate of length will be accurate enough to determine the culvert size.

The culvert length is approximately $2S_eD$ shorter than the distance between the points defined by the intersections of the embankment slopes and the stream bed, where S_e is the embankment slope, and D is the culvert height. The inlet invert elevation will be approximately S_oS_eD lower than the upstream point of intersection and the outlet invert elevation is approximately S_oS_eD higher than the downstream point of intersection, where S_o is the stream bed slope.

All points referenced to the stream bed should be considered approximate since stream beds are irregular and not straight lines as shown in the schematic site representation.

Step 2. Perform Hydrologic Analysis

By hydrologic methods given in Section 3, define the design flow rate.

The probable accuracy of the estimate should be kept in mind as the design proceeds. The accuracy is dependent on the method used to define the flow rate, the available data on which it is based, etc.

Step 3. Perform Outlet Control Calculations and Select Culvert (Charts 4.1 through 4.14)

These calculations are performed before inlet control calculations in order to select the smallest feasible barrel which can be used without the required headwater elevation in outlet control (HW_O) exceeding the allowable headwater elevation (AHW El.). For use in this procedure, the equation for headwater is in terms of elevation.

The full flow outlet control performance curve for a given culvert (size, inlet edge, shape, material) defines its maximum performance. Therefore, inlet improvements beyond the beveled edge or changes in inlet invert elevation will not reduce the required outlet control headwater elevation. This makes the outlet control performance curve an ideal limit for improved inlet design.

When the barrel size is increased, the outlet control curve is shifted to the right, indicating a higher capacity for a given head. Also, it may be generally stated that increased barrel size will flatten the slope of the outlet control curve, although this must be checked.

The outlet control curve passing closest to and below the design Q and AHW El. on the performance curve graph defines the smallest possible barrel which will meet the hydraulic design criteria. However, that curve may be very steep (rapidly increasing headwater requirements for discharges higher than design) or use of such a small barrel may not be practical.

(a) Calculate HW_O at design discharge for trial culvert sizes, entrance condition, shapes, and materials.

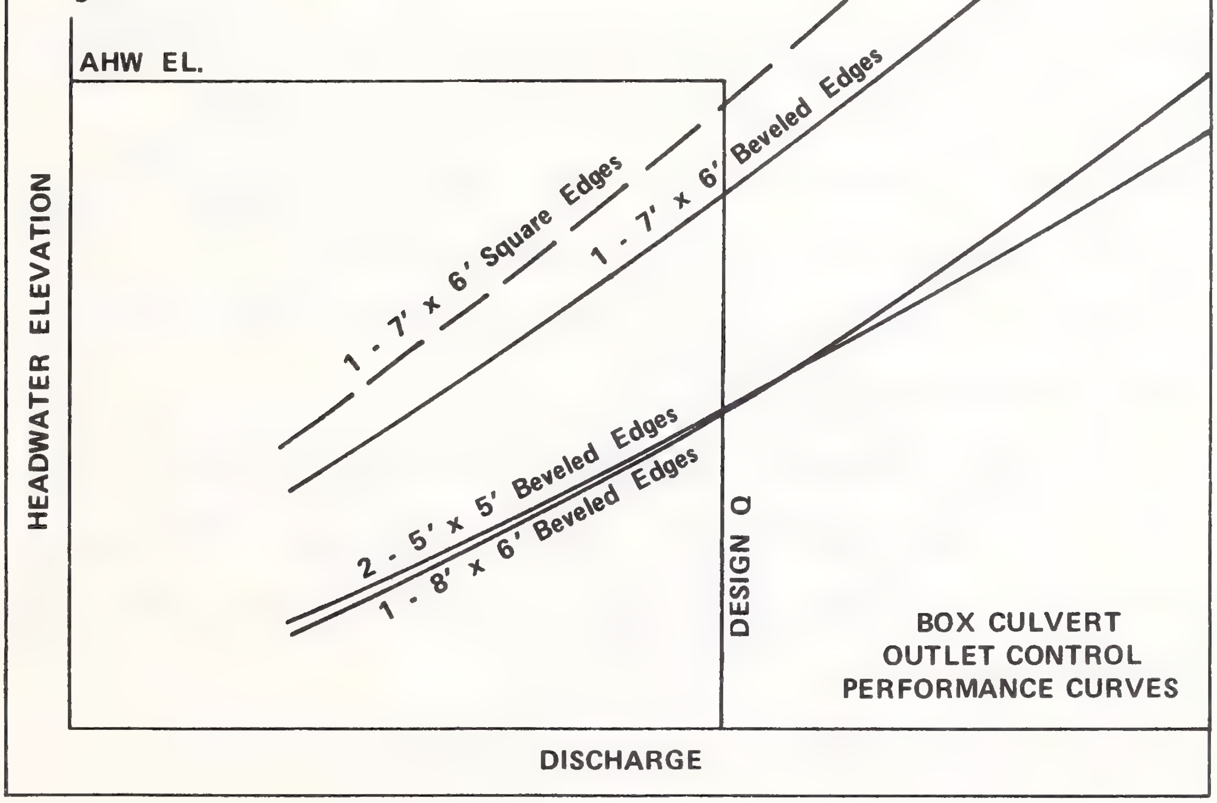
(b) Calculate headwater elevations at two additional discharge values in the vicinity of design Q in order to define outlet control performance.

(c) Plot outlet control performance curves for trial culvert sizes.

(d) Select culvert barrel size, shape, and material.

The selection should not be used solely on calculations which indicate that the required headwater at the design discharge is near the AHW El., but should also be based on outlet velocity as affected by material selection, the designer's evaluation of site characteristics, and the possible consequences of a flood occurrence in excess of the estimated curve may be sufficient reason to select a culvert of different size, shape, or material.

Figure 4.19



In order to zero in on the barrel size required in outlet control, the applicable outlet control nomograph may be used as follows:

(1) Intersect the "Turning Line" with a line drawing between Discharge Head, H. To estimate H, use the following equation:

$$H = \text{AHW El.} - \text{El. Outlet Invert} - h_o$$

where h_o may be selected as a culvert height. Accuracy is not critical

at this point.

(2) Using the point on the "Turning Line", K_e, and the barrel length, draw a line defining the barrel size.

This size gives the designer a good first estimate of the barrel size and more precise sizing will follow rapidly.

Step 4. Perform Inlet Control Calculations for Conventional and Beveled Edge Culvert Inlets (Charts 4.15 through 4.25)

In the calculation procedure note that headwater is defined as an elevation rather than a depth, a FALL may be incorporated upstream of the culvert face, and performance curves are an essential part of the procedure. The depression or FALL should have dimensions as described for side-tapered inlets.

(a) Calculate the required headwater depth (H_f) at the culvert face at design discharge for the culvert selected in Step 3.

(b) Determine required face invert elevation to pass design discharge by subtracting H_f from the AHW El.

(c) If this invert elevation is above the stream bed elevation at the face, invert would generally be placed on the stream bed and the culvert will then have a capacity greater than design Q with headwater as the AHW El.

(d) If this invert elevation is below the stream bed elevation at the face, the invert must be depressed, and the amount of depression is termed the FALL.

(e) Add H_f to the invert elevation to determine HW_f. If HW_f is lower than HW_0, the barrel operates in outlet control at design Q. Proceed to Step 8.

(f) If the FALL is excessive in the designer's judgement from the standpoint of aesthetics, economy and other engineering reasons, a need for inlet geometry refinements is indicated. If square edges

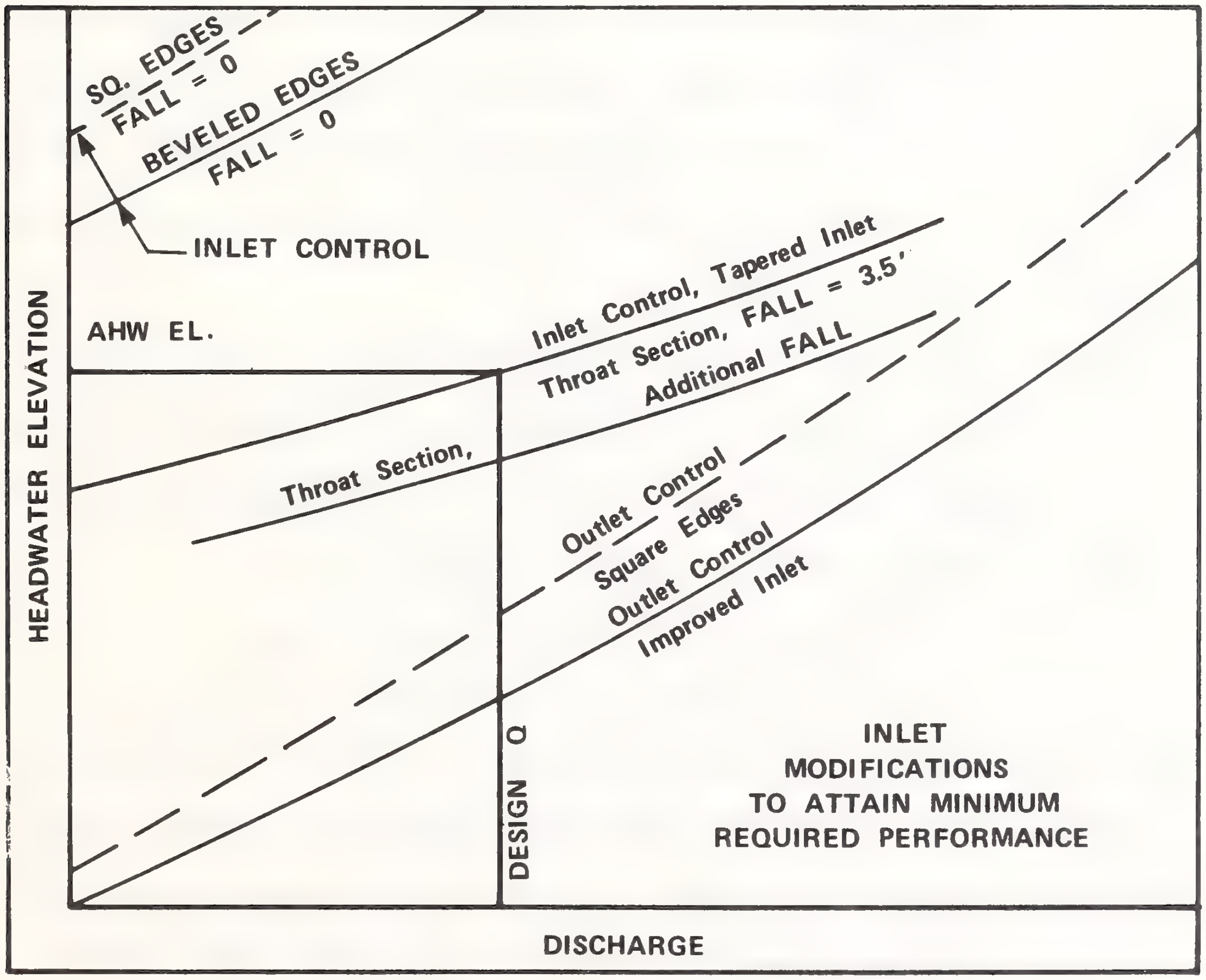
were used in Steps 3 and 4 above, repeat with beveled edges. If beveled edges were used, proceed to Step 5.

(g) If the FALL is within acceptable limits, determine the inlet control performance by calculating required headwater elevation using the flow rates from Step 3 and the FALL determined above. $HW_f = H_f + El.$ face invert.

(h) Plot the inlet control performance curve with the outlet control performance curve plotted in Step 3.

(i) Proceed to Step 6.

Figure 4.20



Step 5. Perform Throat Control Calculations for Side- and Slope-Tapered Inlets (Charts 4.26 or 4.30)

The same concept is involved here as with conventional or beveled edge culvert design.

(a) Calculate required headwater depth on the throat (H_t) at design Q for the culvert selected in Step 3.

(b) Determine required throat elevation to pass design discharge by subtracting H_t from the AHW El.

(c) If this throat invert elevation is above the stream bed elevation, the invert would probably be placed on the stream bed and the culvert throat will have a capacity greater than the design Q with headwater at the AHW El.

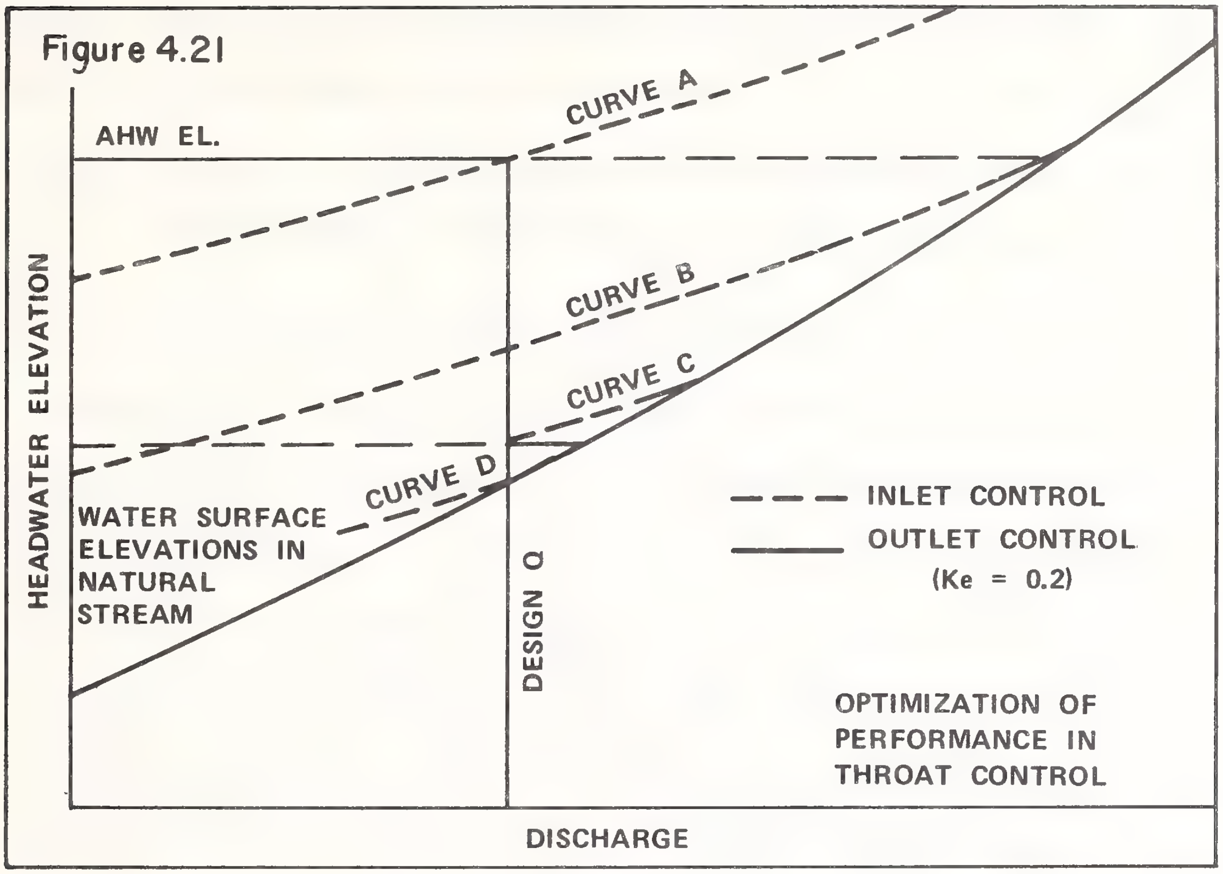
(d) If this throat invert elevation is below the stream bed elevation, the invert must be depressed, and the elevation difference between the stream bed at the face and the throat invert is termed the FALL. If the FALL is determined to be excessive, a larger barrel must be selected. Return to Step 5(a).

(e) Add H_t to the invert elevation to determine HW_t. If HW_t is lower than HW_o, the culvert operates in outlet control at design Q. In this case, adequate performance can probably be achieved by the use of beveled edges with a FALL. Return to Step 4.

(f) Define and plot the throat control performance curve.

Step 6. Analyze the Effect of FALLS on Inlet Control Section Performance

It is apparent from Figure 4.20 that either additional FALL or inlet improvements would increase the culvert capacity in inlet control by moving the inlet control performance curve to the right toward the outlet control performance curve. If the outlet control performance curve of the selected culvert passes below the point defined by the AHW El. and the design Q, there is an opportunity to optimize the culvert design by selecting the inlet so as to either increase its capacity to the maximum at the AHW El. or to pass the design discharge at the lowest possible headwater elevation.



Some possibilities are illustrated in Figure 4.21. The minimum inlet control performance which will meet the selected design criteria is illustrated by Curve A. This design has merit in that minimum expense for inlet improvements and/or FALL is incurred and the inlet will pass a flood in excess of design Q before performance is adequate in many location, inlcuding those locations where headwaters in excess of the AHW El. would be tolerable on the rare occasion of floods in excess of design Q.

Curve B illustrates the performance of a design which takes full advantage of the potential capacity of the selected culvert and the site to pass the maximum possible flow at the AHW El. A safety factor in capacity is thereby incorporated by geometry improvements at the inlet or by a combination of the two. Additional inlet improvements and/or FALL will not increase the capacity at or above the AHW El.

There may be reason to pass the design flow at the lowest possible headwater elevation even though the reasons are sufficient to cause the AHW El. to be set at a lower elevation. The maximum possible reduction in headwater at design Q is illustrated by Curve C. Additional inlet improvement and/or FALL will not reduce the required headwater elevation at design Q.

The water surface elevation in the natural stream may be a limiting factor in design, i.e., it is not productive to design for headwater at a lower elevation than natural stream flow elevations. The reduction in headwater elevation illustrated by Curve C is limited by natural water surface elevations in the stream. If the water surface elevations in the natural stream had fallen below Curve D, this curve would illustrate the maximum reduction in headwater elevation at design Q. Tailwater depths calculated by assuming normal depth in the stream channel may be used to estimate natural water surface elevations in the stream at the culvert inlet. These may have been computed as a part of Step 3.

Curve A has been established in either Step 4 for conventional culverts or Step 5 for improved inlets. To define any other inlet control performance curve such as B, C, or D for the same control section:

(a) Select a point on the outlet control performance curve.

(b) Measure the vertical distance from this point to Curve A. This is the difference in FALL between Curve A and the curve to be established, e.g., the FALL on the control section for Curve A plus the distance between Curves A and B is the FALL on the control section for Curve B.

For conventional culverts only:

(d) Estimate and compare the costs incurred for FALLS (structural excavation and additional culvert length) to achieve various levels of inlet performance.

(e) Select design with increment in cost warranted by increased capacity

and improved performance.

(f) If Fall required to achieve desired performance is excessive, proceed to Step 5.

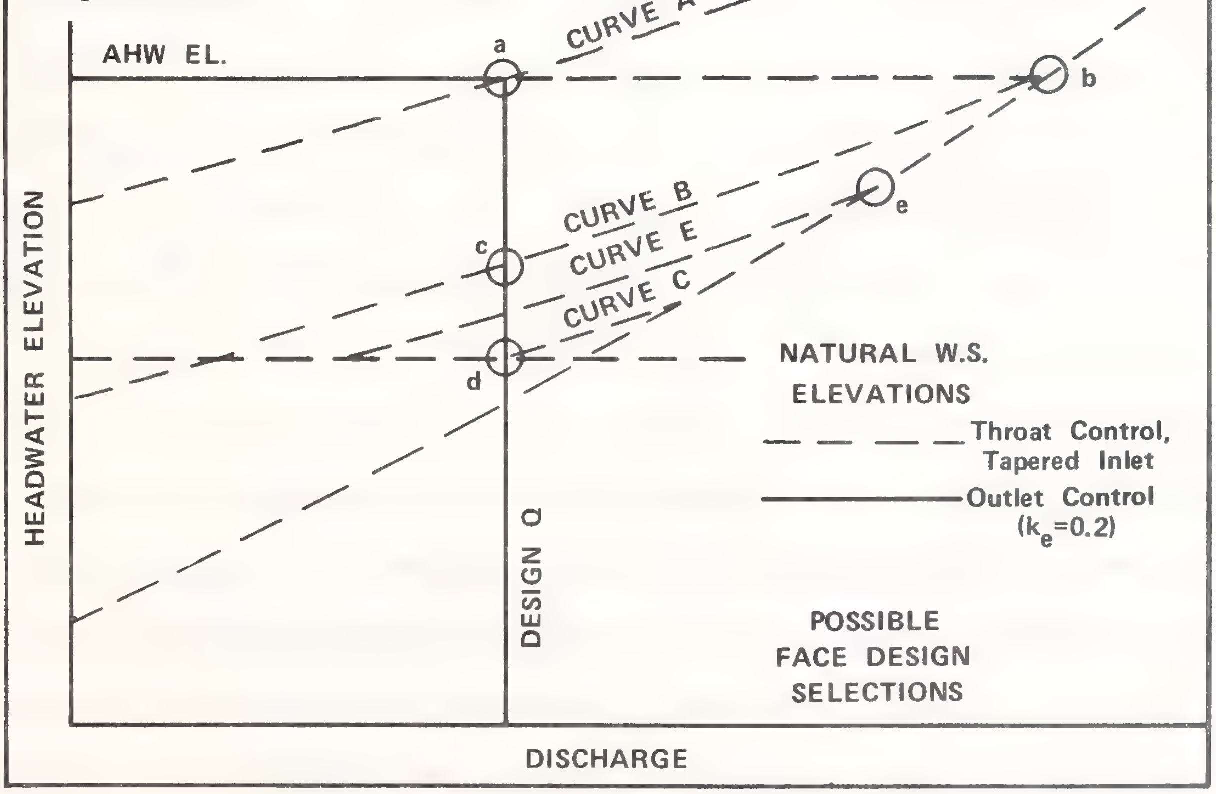
(g) If FALL is acceptable and performance achieves the design objectives, proceed to Step 8.

Step 7. Design Side-and/or Slope-Tapered Inlet (Charts 4.28, 4.29, and 4.31).

Either a side- or slope-tapered inlet design may be used if a FALL is required on the throat by use of a depression (FALL) upstream of the face of a side-tapered inlet or a FALL in the inlet of a slope-tapered inlet.

The face of the side- or the slope-tapered inlet should be designed to be compatible with the throat performance defined in Step 6. The basic principles of selecting the face design are illustrated in Figure 4.22.

Figure 4.22



The minimum face design is one whose performance curve does not exceed the AHW El. at design Q. However, a "balanced" design requires that full advantage be taken of the increased capacity and/or lower headwater requirement gained through use of various FALLS. This suggests a face performance curve which intersects the throat control curve: (1) at the AHW El., (2) at design Q, (3) at its intersection with the outlet control curve, or (4) other. These options are illustrated in Figure 4.22 by points a through e representing the intersections of face control performance curves with the throat control performance curves. The options are explained as follows: (1) Intersection of face and throat control performance curves at the AHW El. (Point a or b): For the minimum acceptable throat control performance (Curve A), this is the minimum face size that can be used without the required headwater elevation (HW_f) exceeding the AHW El. at design Q (Point a). For throat control performance greater than minimum but equal to or less than Curve B., this is the minimum face design which makes full use of the FALL placed on the throat to increase culvert capacity at the AHW El. (Point b), (2) Intersection of face and throat control performance curves at design Q (Points a, c, or d): This face design option results in throat control performance at discharges equal to or greater than design Q. It makes full use of the FALL to increase capacity and reduce headwater requirements at flows equal to or greater than the design Q. (3) Intersection of the face control performance curve with the throat control performance curve (Points b or e): This option is the minimum face design which can be used to make full use of the increased capacity available from the FALL placed on the throat. It cannot be used where HW_f would exceed AHW El. at design Q; e.g., with the minimum acceptable throat control performance curve. (4) Other; Variations in the above options are available to the designer. The culvert face can be designed so that culvert performance will change from face control to throat control at any discharge at which inlet control governs. Options (1) through (3), however, appear to

fulfill design objectives of minimum face size to achieve the maximum increase in capacity possible for a given FALL, or the maximum possible decrease in the required headwater for a given FALL for any discharge equal to or greater than design Q.

Figure 4.23

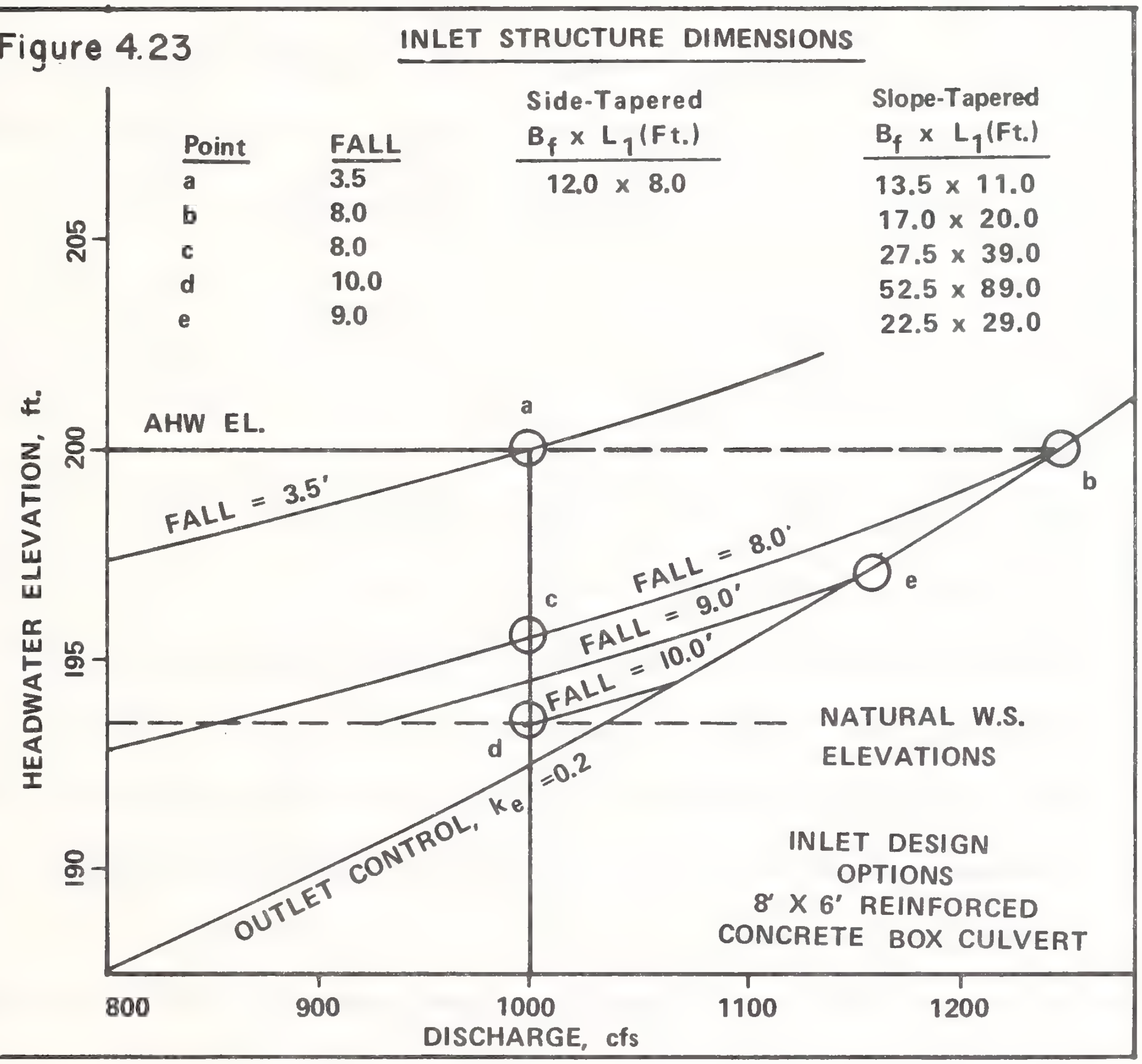


Figure 4.23 illustrates the optional tapered inlet designs possible. Note that the inlet dimensions for the side-tapered inlet are the same for all options. This is because performance of the side-tapered inlet nearly parallels the performance of the throat and an increase in headwater on the throat by virtue of an increased FALL results in an almost equal increase in headwater on the face. Each foot of FALL on the throat of a culvert with a side-tapered inlet requires additional barrel length equal to the fill slope; e.g., if the

fill slope is 3:1, use of 4 ft. of FALL rather than 3 ft. results in a culvert barrel 3 ft. longer as well as increased culvert capacity and/or reduced headwater requirements.

Face dimensions and inlet length increase for the slope-tapered inlet as the capacity of the culvert is increased by additional FALL on the throat. No additional head is created for the face by placing additional FALL on the throat. On the other hand, use of a greater FALL at the throat of a culvert with a slope-tapered inlet does not increase culvert length.

The steps followed in the tapered inlet designs are:

(a) Compute H_f for side - and slope-tapered inlets for various FALLS at design Q and other discharges.

Side-Tapered Inlet: $H_f = H_t - 1.o'$ (Approximate)

Slope-Tapered Inlet: H_f = HW El. - Stream bed El. at face.

(b) Determine dimensions of side- and slope-tapered inlets for trial options.

(c) For slope-tapered inlets with mitered face, check for crest control.

(d) Compare construction costs for various options, including the cost of FALL on the throat.

(e) Select design with incremental cost warranted by increased capacity and improved performance.

From the above, it is apparent that in order to optimize culvert design, performance curves are an integral part of the design procedure. At many culvert sites, designers have valid reasons for providing a safety factor in designs. These reasons include uncertainty in the design discharge estimate, potentially disastrous results in property damage or damage to the highway from headwater elevations which exceed the allowable, the potential for development upstream during the life to the installation. Quantitative analysis of these variables would amount to a risk analysis, but at present, many of these factors must be

evaluated intuitively. Procedures described here enables the designer to maximum the performance of the selected culvert or to optimize the design in accordance with his evaluation of site constraints, design parameters, and costs for construction and maintenance.

Step 8. Complete File Documentation

Documentation of the culvert hydraulic design consists of the compilation and preservation of all hydrologic and hydraulic information and the design decisions made on the basis of this information. This information should include culvert design calculations, all survey notes, all labeled photographs and a copy of the Form HYD-1. Several decisions in the design procedure are based on the designer's knowledge and evaluation of site conditions. These decisions should be well founded on field information and documented for future reference.

Each decision regarding culvert performance should be made with knowledge of the accuracy of the flood estimate and an understanding that, even though the accuracy of the estimate may be relatively good, there is a chance that the design frequency event will be exceeded during the life of the project. The files should reflect the basis of the design flood estimate, the designer's evaluation of the goodness of the estimate, the consideration given to consequences of a flood occurrence in excess of the design flooding. This documentation can be of inestimable value in evaluating the performance of highway culverts after large floods, or, in the event of failure, in identifying contributing factors. It also will provide valuable information for use in the event that flood damage claims are made of the department following construction of the highway.

Adequate documentation of the design decisions which were made and the above basic information on which those decisions were based should be placed in the files to support all hydraulic structure designs. The completeness

of documentation needed to support designs will vary with the importance of the structure, but structure costs should not be the sole basis for this determination. The potential for loss of property and life, traffic interruption, the importance of the highway and the availability of alternate routes are among the factors that should be considered in making this determination.

Documentation should be kept in the Hydraulic Unit's files so that the performance of the designs they represent can be used as a foundation for better designs in the future.

DIMENSIONAL LIMITATIONS

Side-Tapered Inlets

1. $6:1 \geq$ Taper $\geq 4:1$

   Tapers greater than 6:1 may be used but performance will be underestimated.

2. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

3. If FALL is used upstream of face, extend barrel invert slope upstream from face a distance of D/2 before sloping upward more steeply.

4. For pipe culverts, these additional requirements apply:

   a. $D \leq E \leq 1.1D$

   b. Length of square to round transition $\geq 0.5D$

   c. FALL (Figure 4.16)

      $P \geq 3T$

      $W_p = B_f + T$ or $4T$, whichever is larger.

Slope-Tapered Inlets

1. $6:1 \geq$ Taper $\geq 4:1$

   Tapers $> 6:1$ may be used, but performance will be underestimated.

2. $3:1 \geq S_f \geq 2.1$

   If $S_f > 3:1$, use side-tapered design.

3. Minimum L_3 = 0.5B

4. 1.5D $\geq$ FALL $\geq$ D/4

   For FALL $<$ D/4, use side-tapered design

   For FALL $>$ 1/5D, estimate friction losses between face and throat.

5. Wingwall flare angle from 15 degrees to 26 degrees with top edge beveled or from 26 degrees to 90 degrees with or without bevels.

6. For pipe culverts, these additional requirements apply:

   a. Square to circular transition length $>$ 0.5D

   b. Square throat dimension equal to barrel diameter. Not necessary to check square throat performance.

DESIGN TABLES AND CHARTS

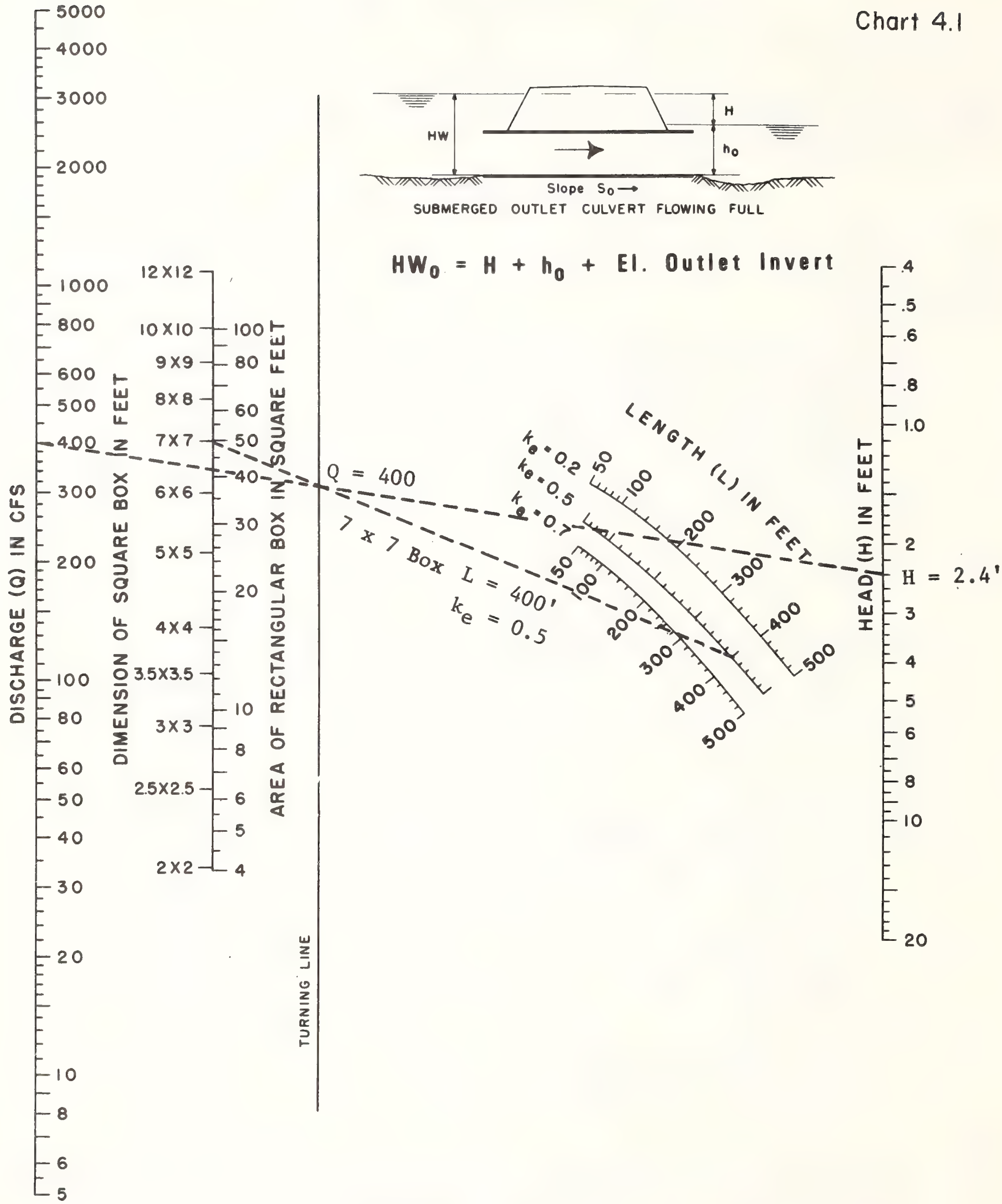
TABLE 4.1 - ENTRANCE LOSS COEFFICIENTS

Coefficient k_e to apply to velocity head $\frac{V^2}{2g}$ for determination of head loss at entrance to a structure, such as a culvert or conduit, operating full or partly full with control at the outlet.

$$\text{Entrance head loss } H_e = K_e \frac{V^2}{2g}$$

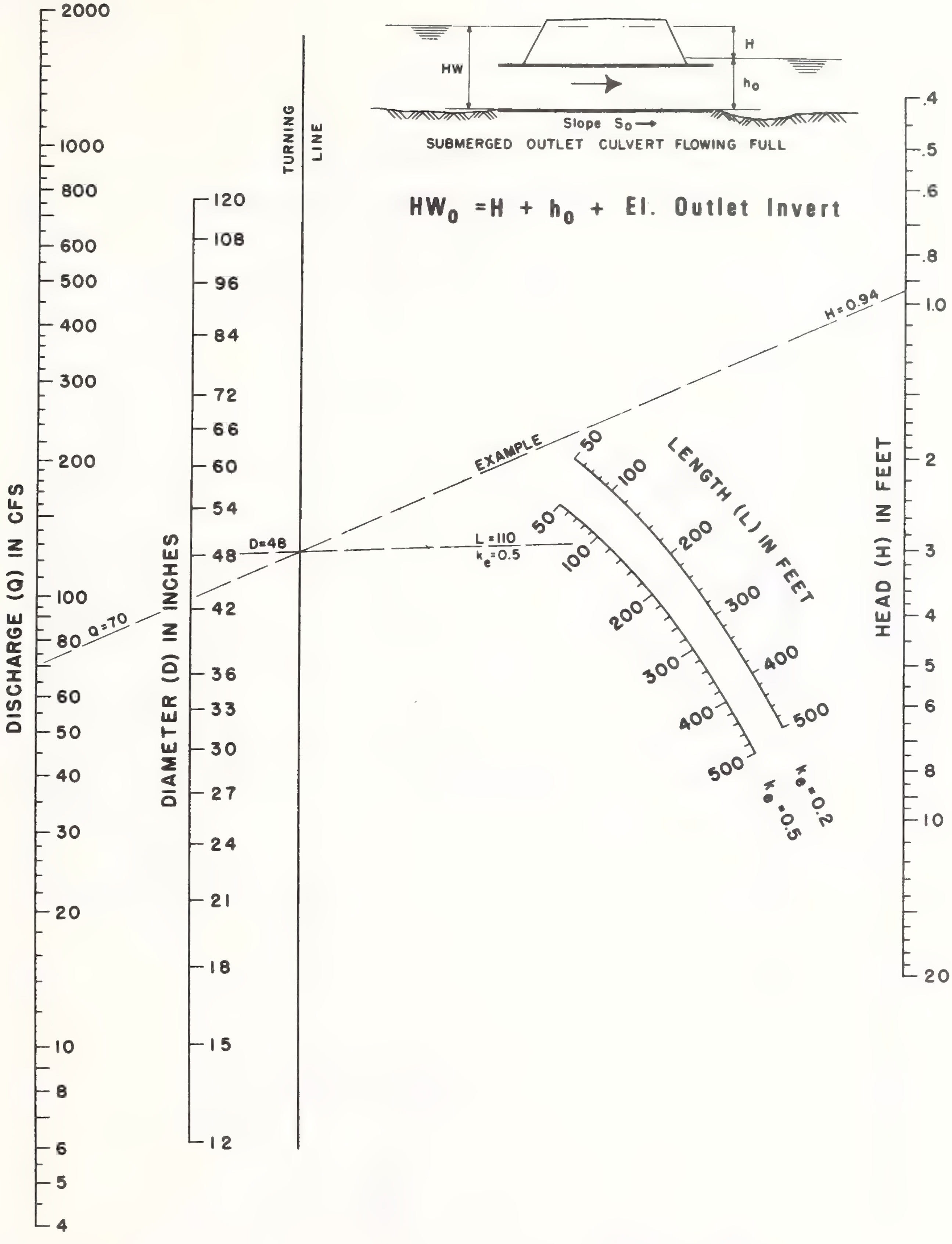
Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (groove end) . . .	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded (radius - 1/12D).	0.2
Mitered to conform to fill slope.	0.7
*End-Section conforming to fill slope	0.5
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall).	0.9
Headwall or headwall and wingwalls	
Square-edge	0.5
Mitered to conform to fill slope.	0.7
*End-Section conforming to fill slope	0.5
Box, Reinforced Concrete	
Headwall parallel to embackment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7

*Note: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections, incorporating a closed taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet, p.



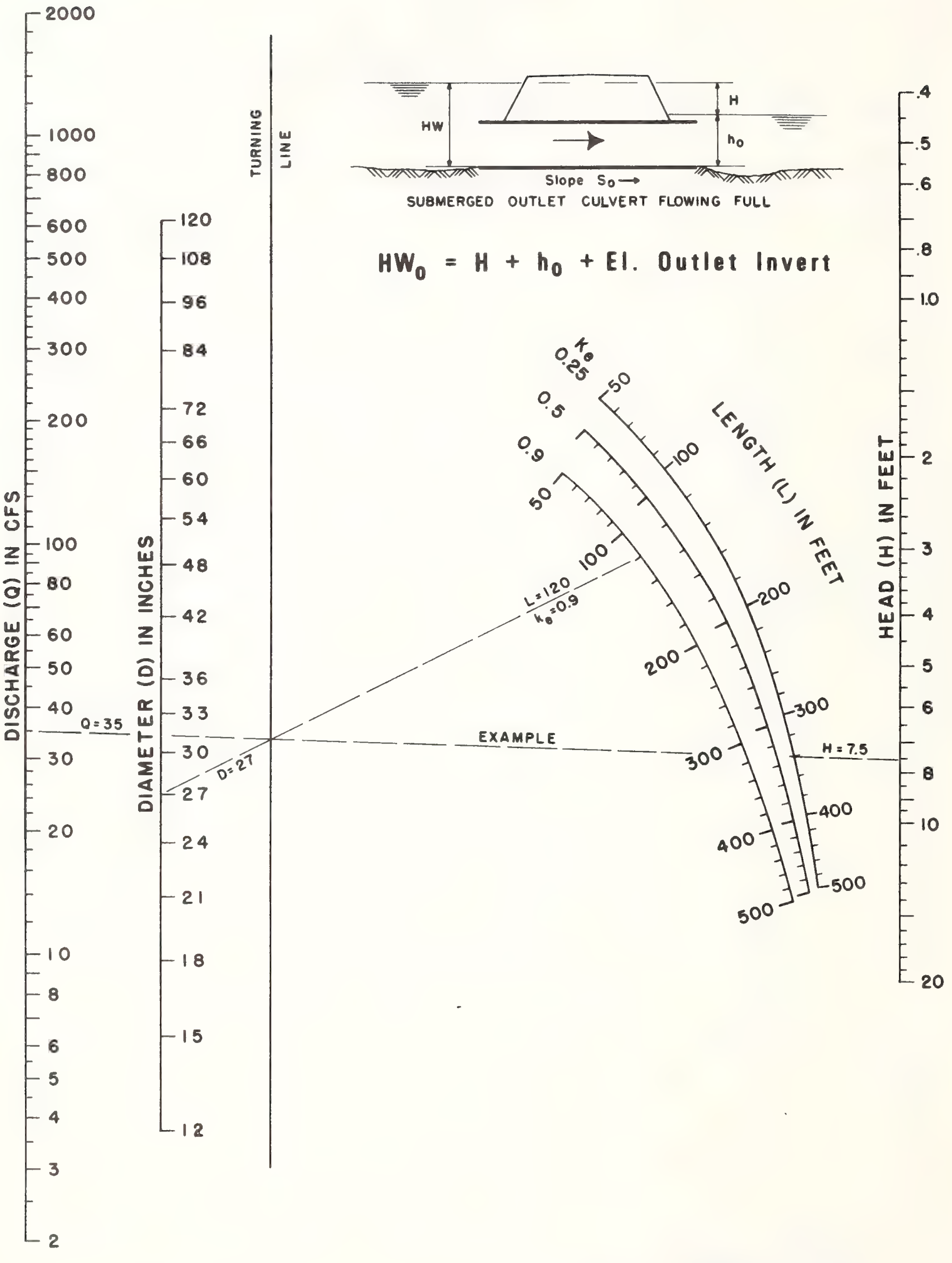
HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963



HEAD FOR STANDARD C. M. PIPE CULVERTS FLOWING FULL
n = 0.024

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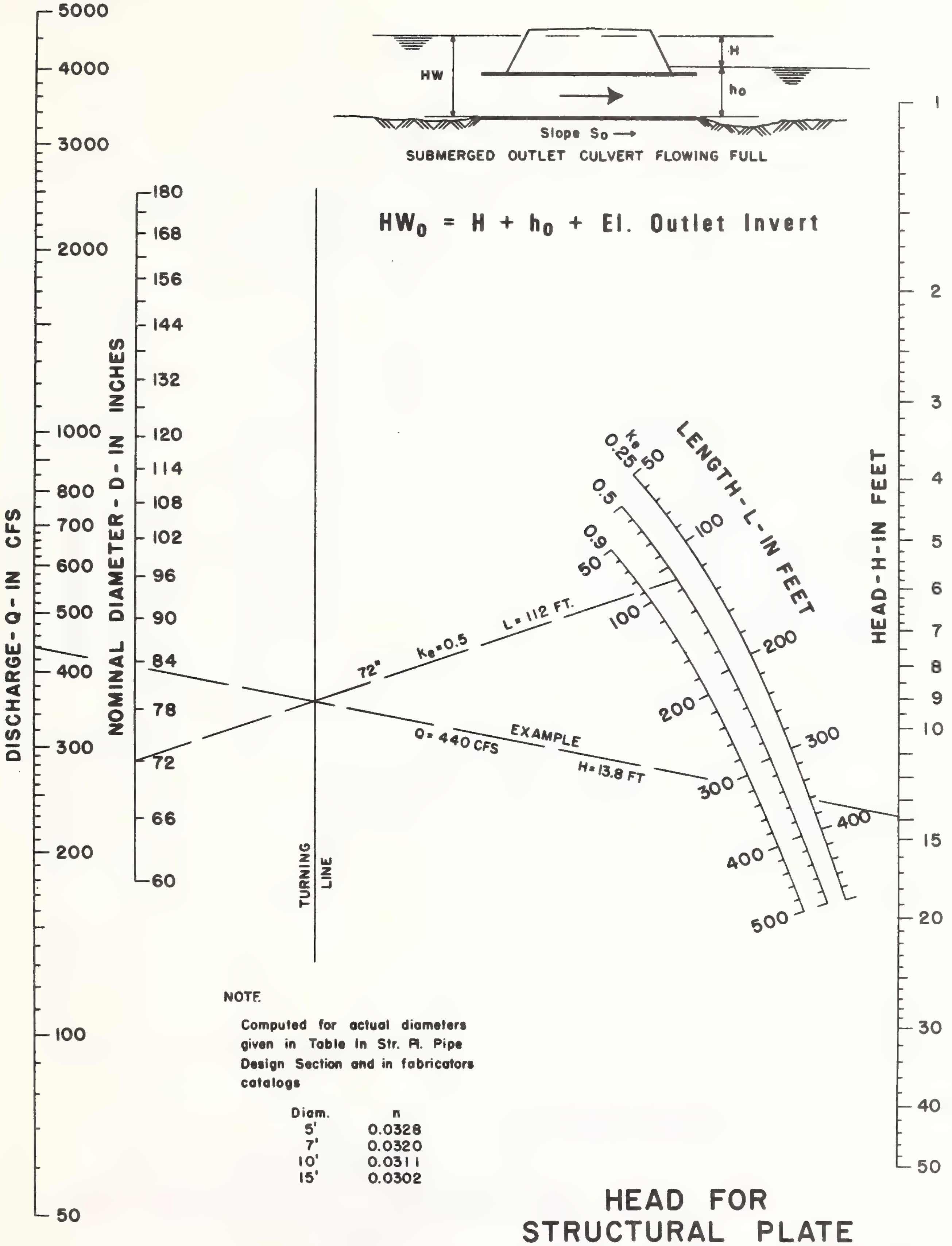

NOTE

Computed for actual diameters given in Table In Str. Pl. Pipe Design Section and in fabricators catalogs

Diam.	n
5'	0.0328
7'	0.0320
10'	0.0311
15'	0.0302

HEAD FOR STRUCTURAL PLATE CORR. METAL PIPE CULVERTS FLOWING FULL $n = 0.0328$ TO 0.0302

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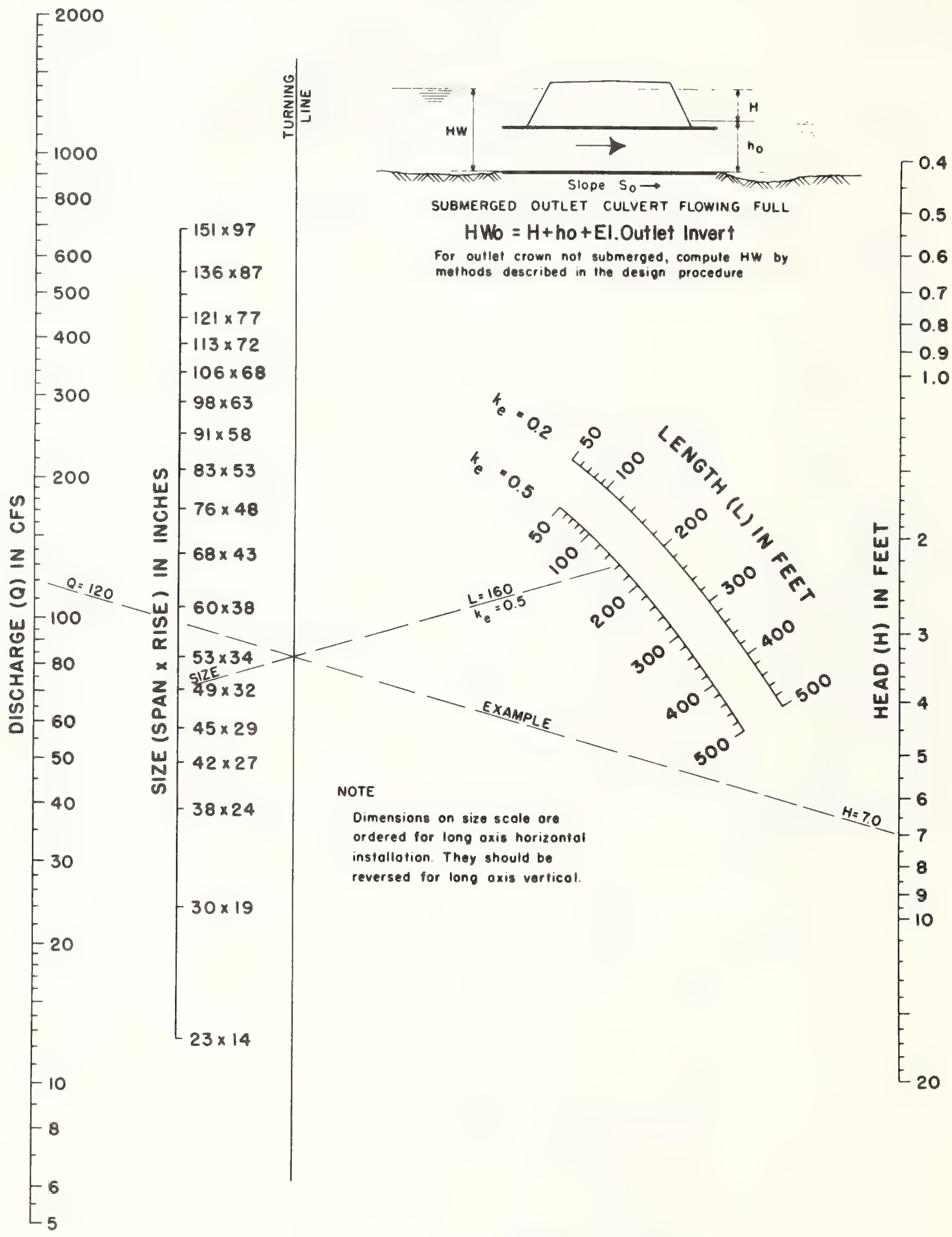
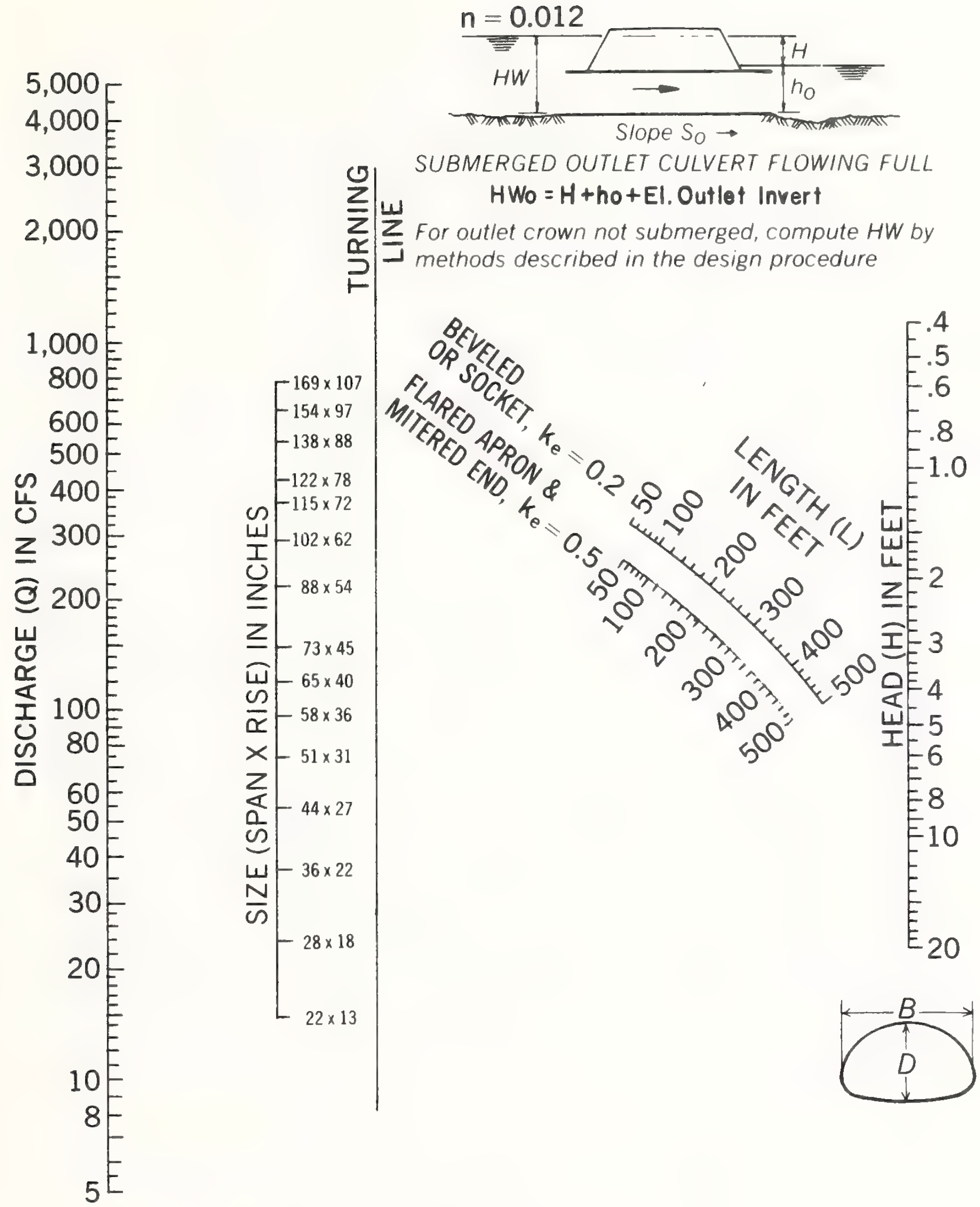


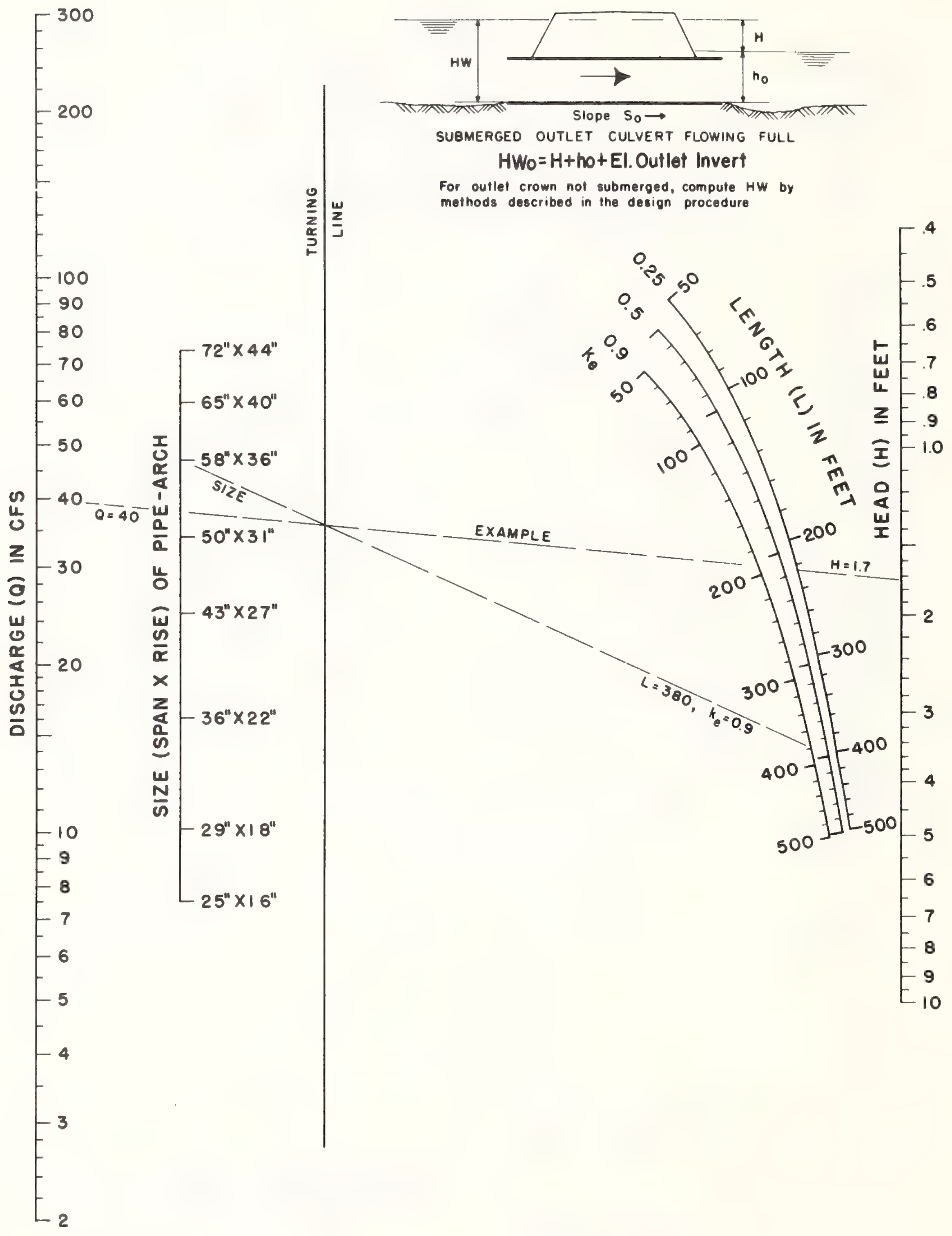
HEAD FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL OR VERTICAL
FLOWING FULL
$n = 0.012$

BUREAU OF PUBLIC ROADS JAN. 1963



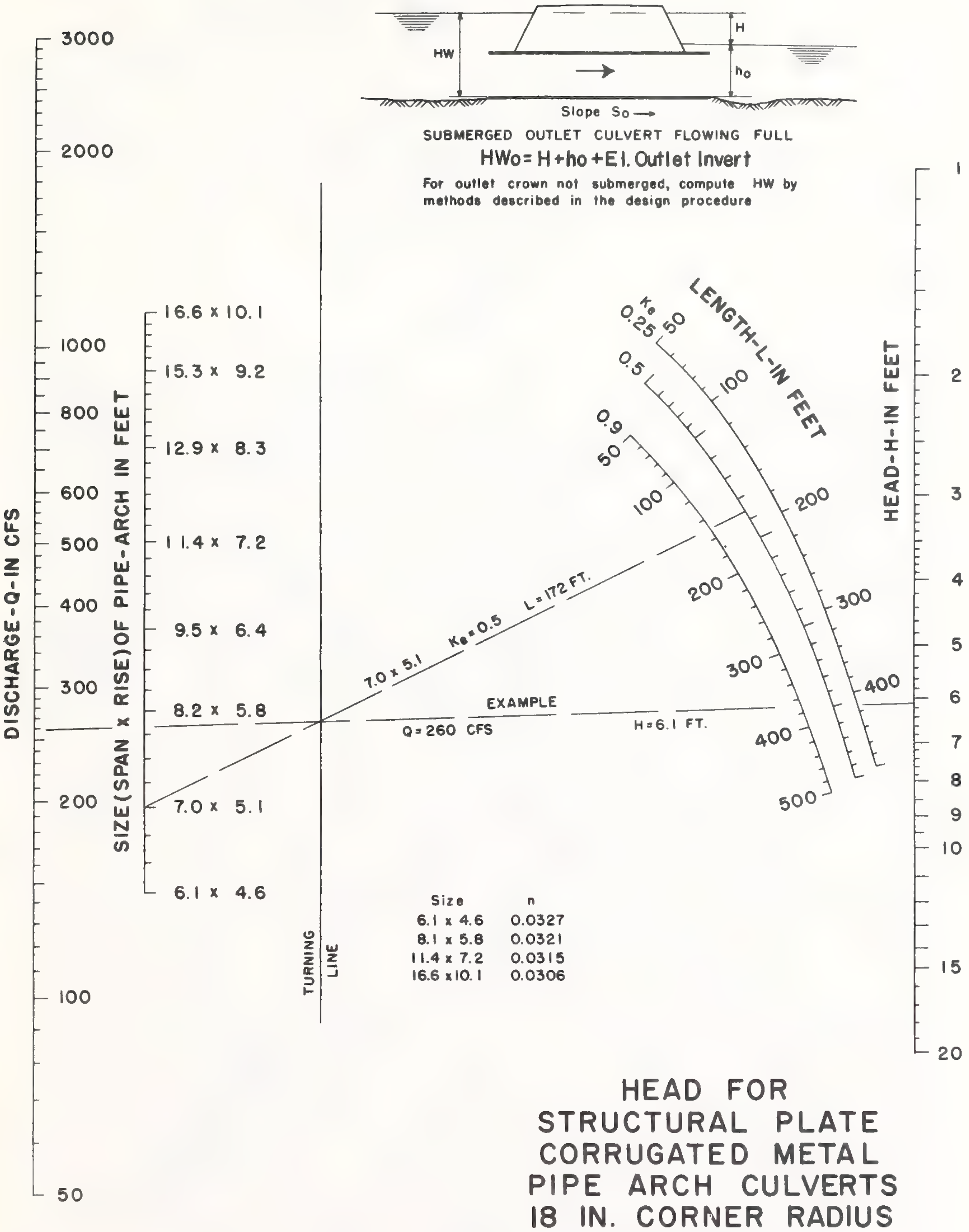
American Concrete Pipe Association 1970

HEAD FOR CONCRETE ARCH CULVERTS FLOWING FULL n=.012



HEAD FOR STANDARD C. M. PIPE-ARCH CULVERTS FLOWING FULL
n=0.024

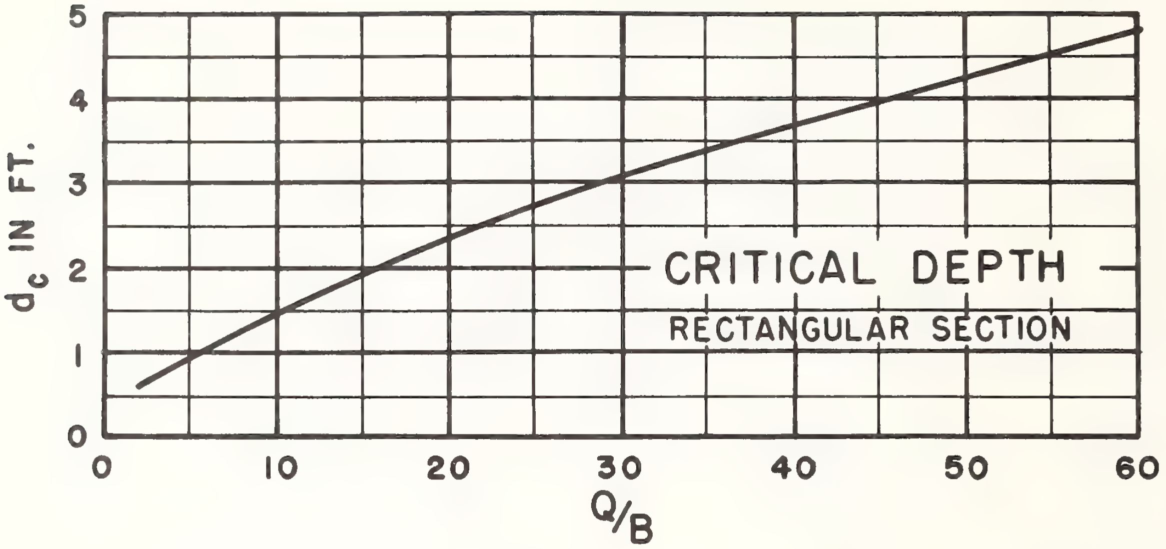
BUREAU OF PUBLIC ROADS JAN. 1963

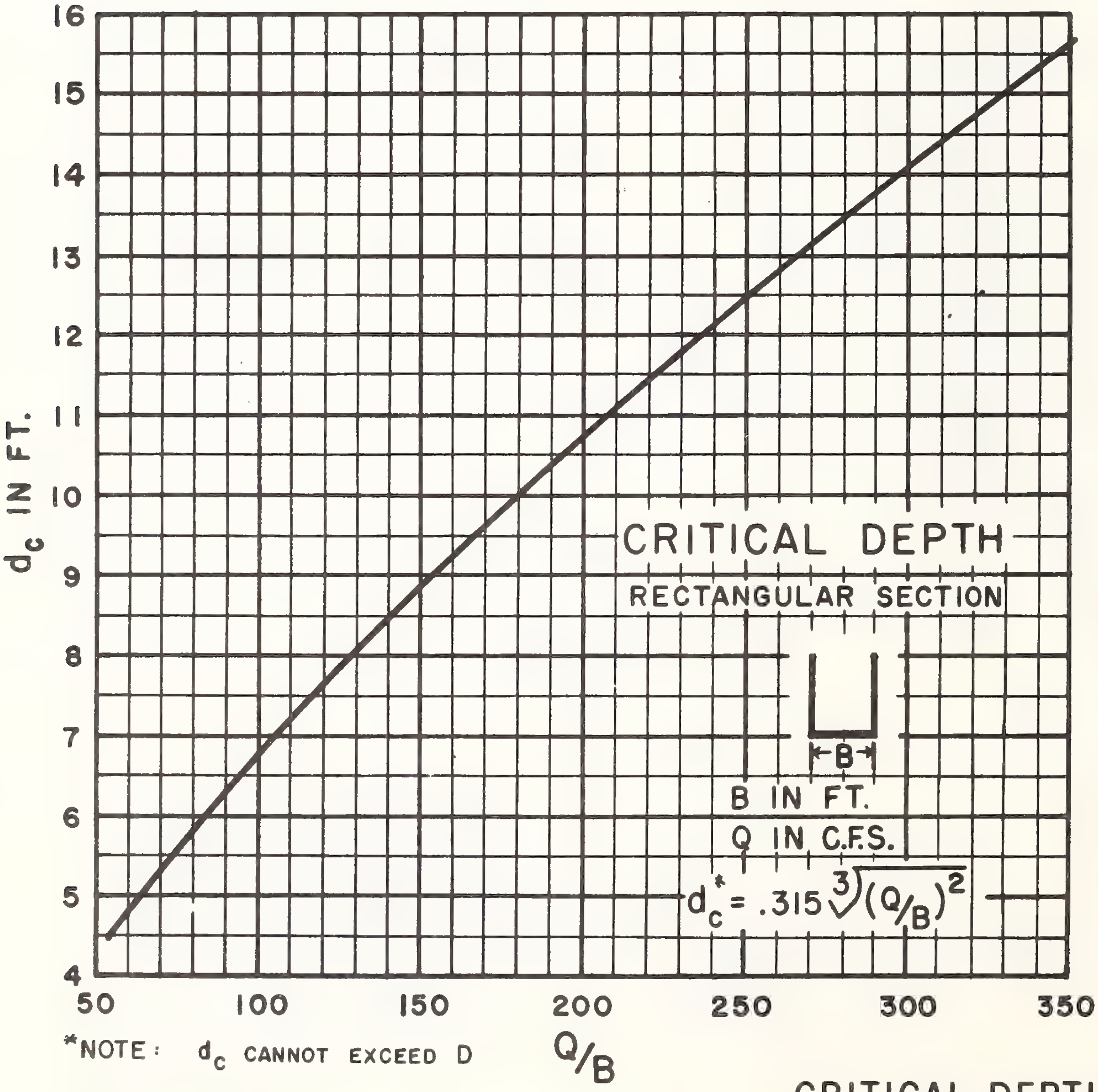


Size	n
6.1 x 4.6	0.0327
8.1 x 5.8	0.0321
11.4 x 7.2	0.0315
16.6 x 10.1	0.0306

HEAD FOR STRUCTURAL PLATE CORRUGATED METAL PIPE ARCH CULVERTS 18 IN. CORNER RADIUS FLOWING FULL n=0.0327 TO 0.0306

BUREAU OF PUBLIC ROADS JAN. 1963





*NOTE: d_c CANNOT EXCEED D

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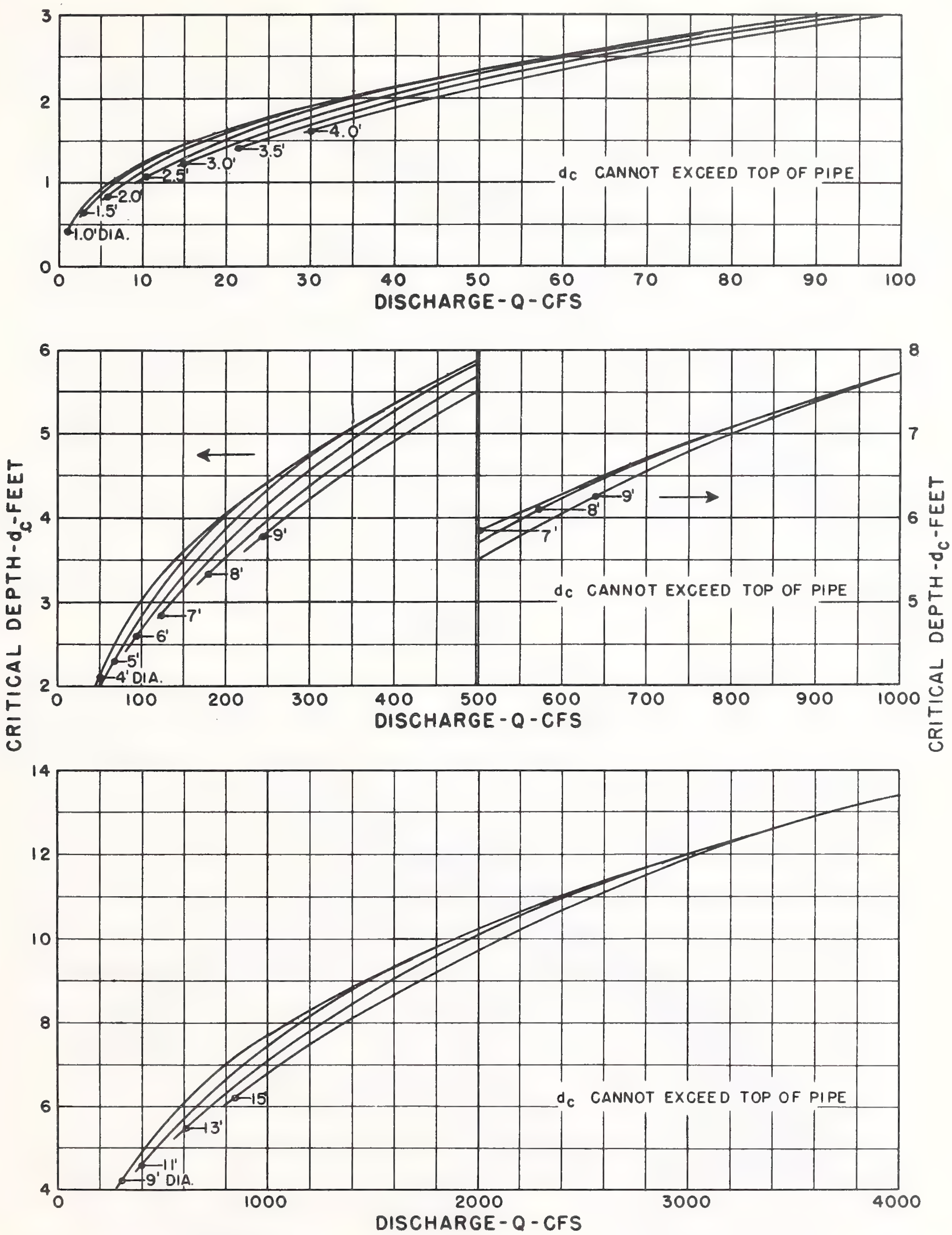
CRITICAL DEPTH
RECTANGULAR SECTION

Chart 4.10



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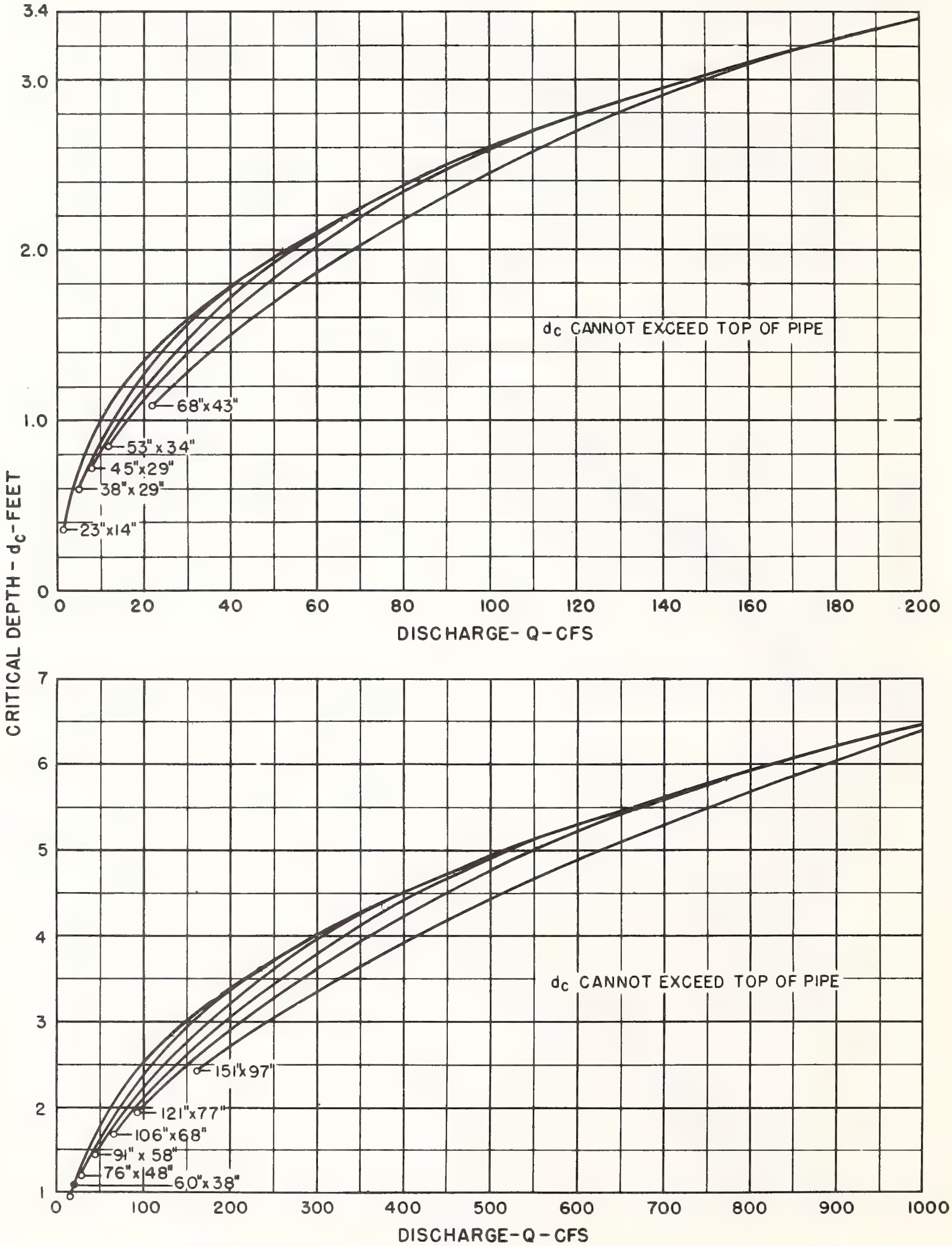
CRITICAL DEPTH
CIRCULAR PIPE



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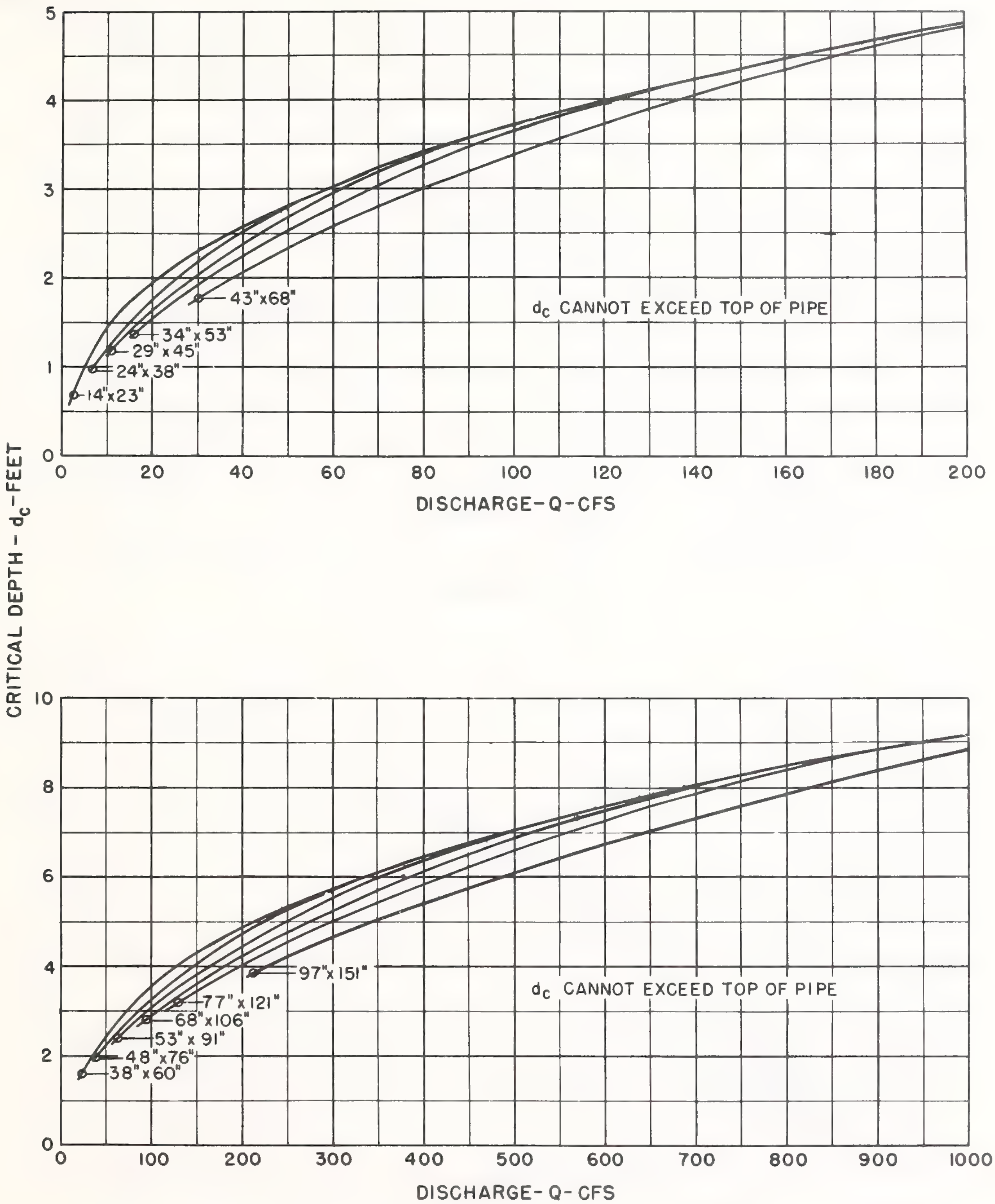
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL



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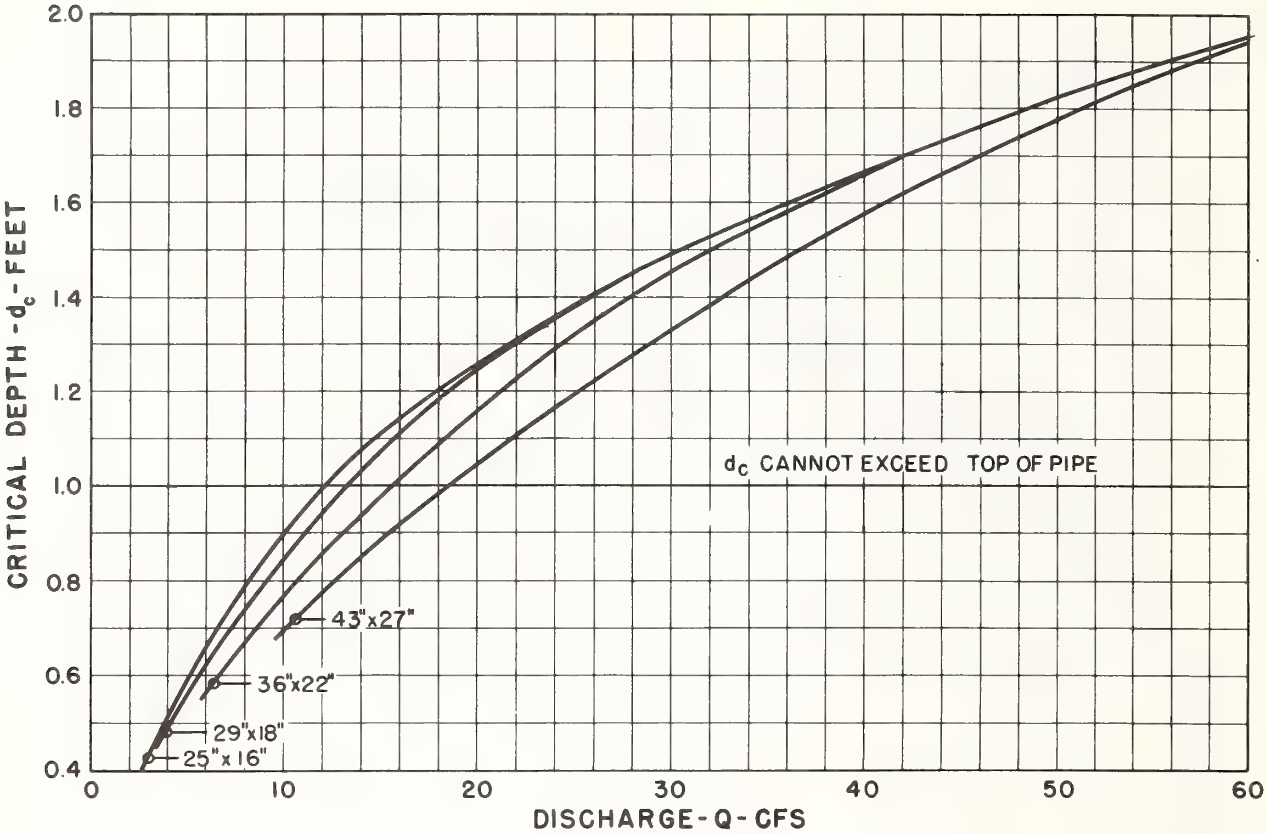
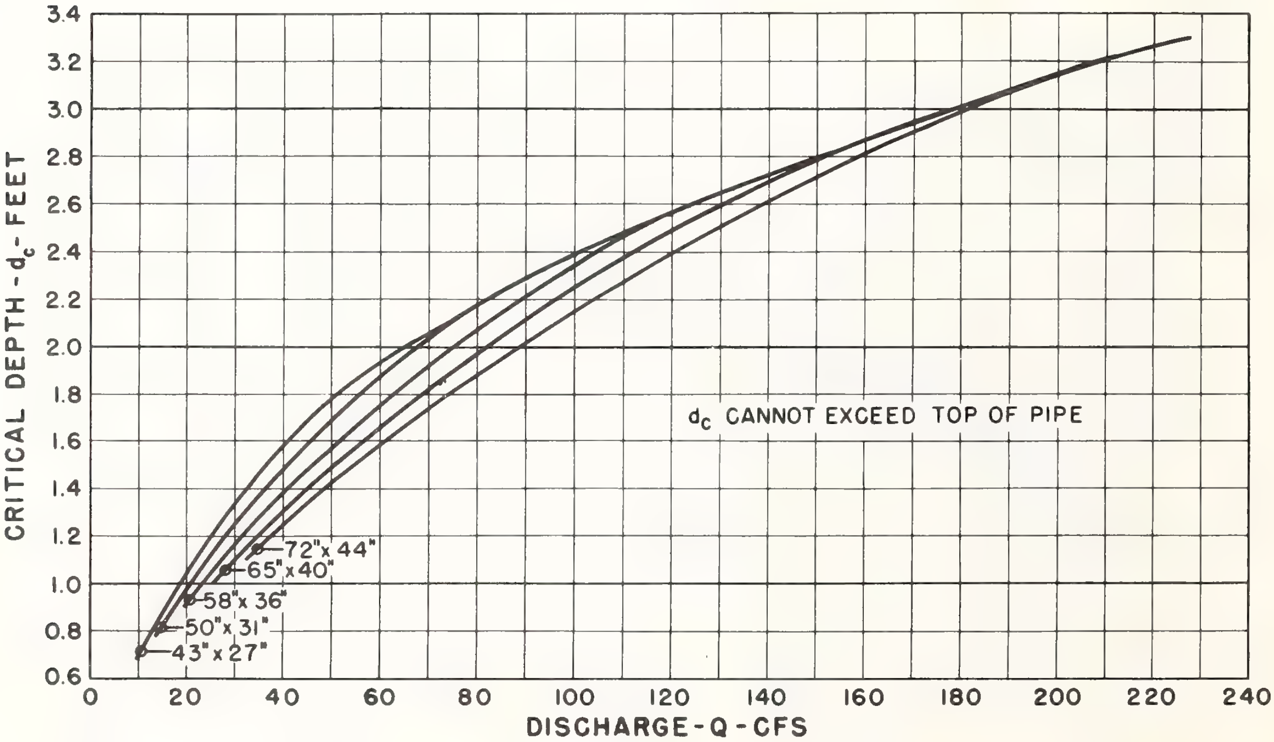
CRITICAL DEPTH
OVAL CONCRETE PIPE
LONG AXIS VERTICAL





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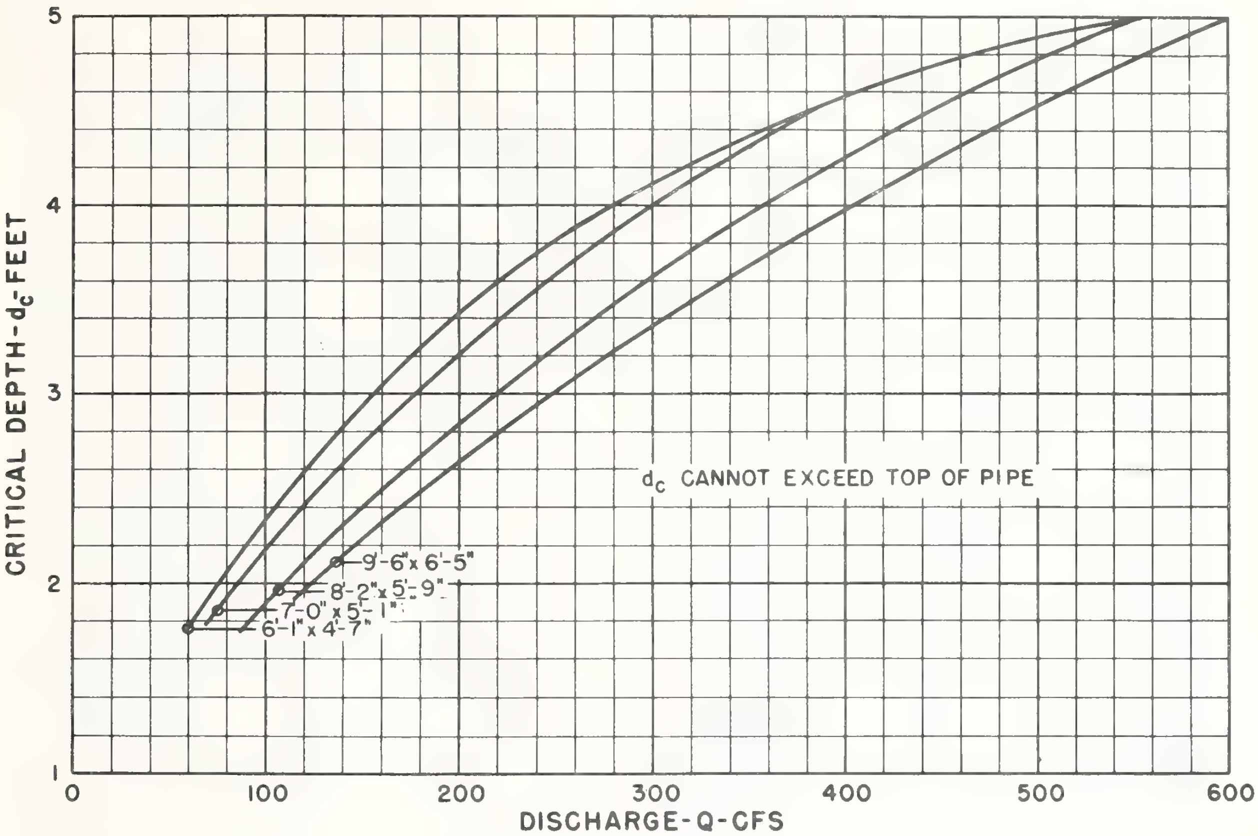
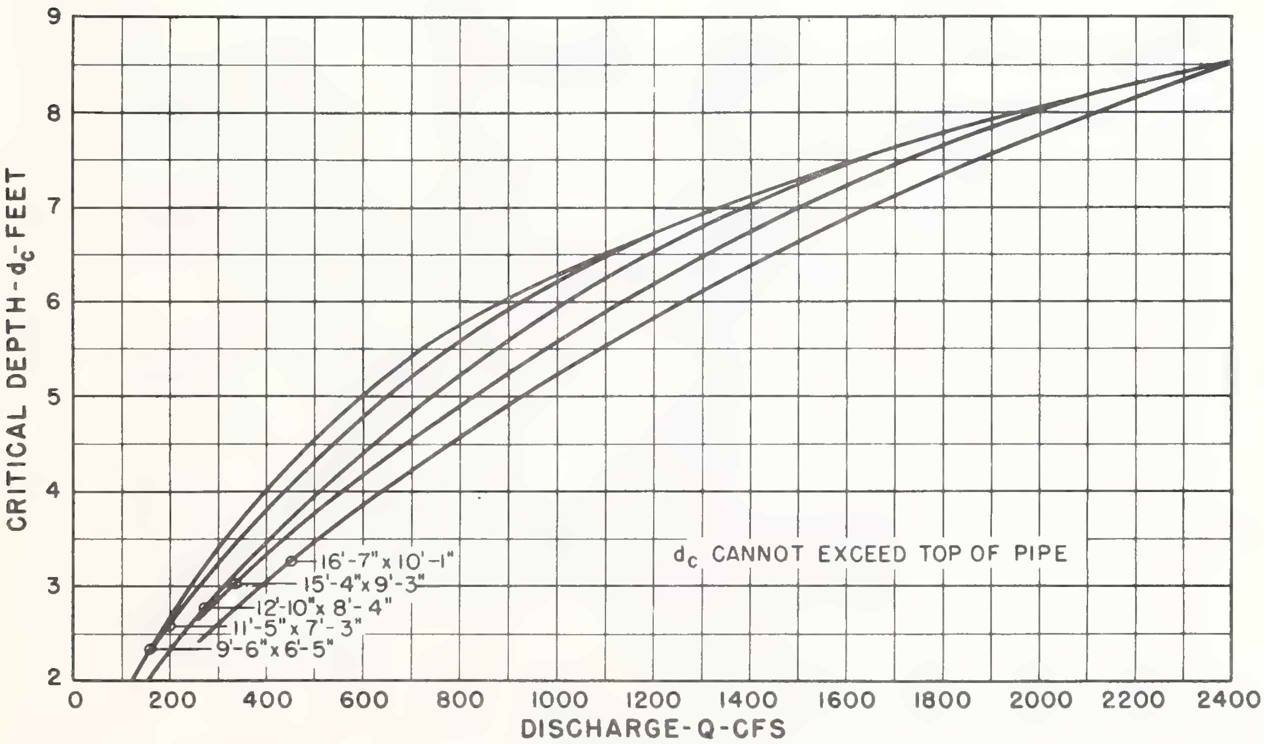
CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH





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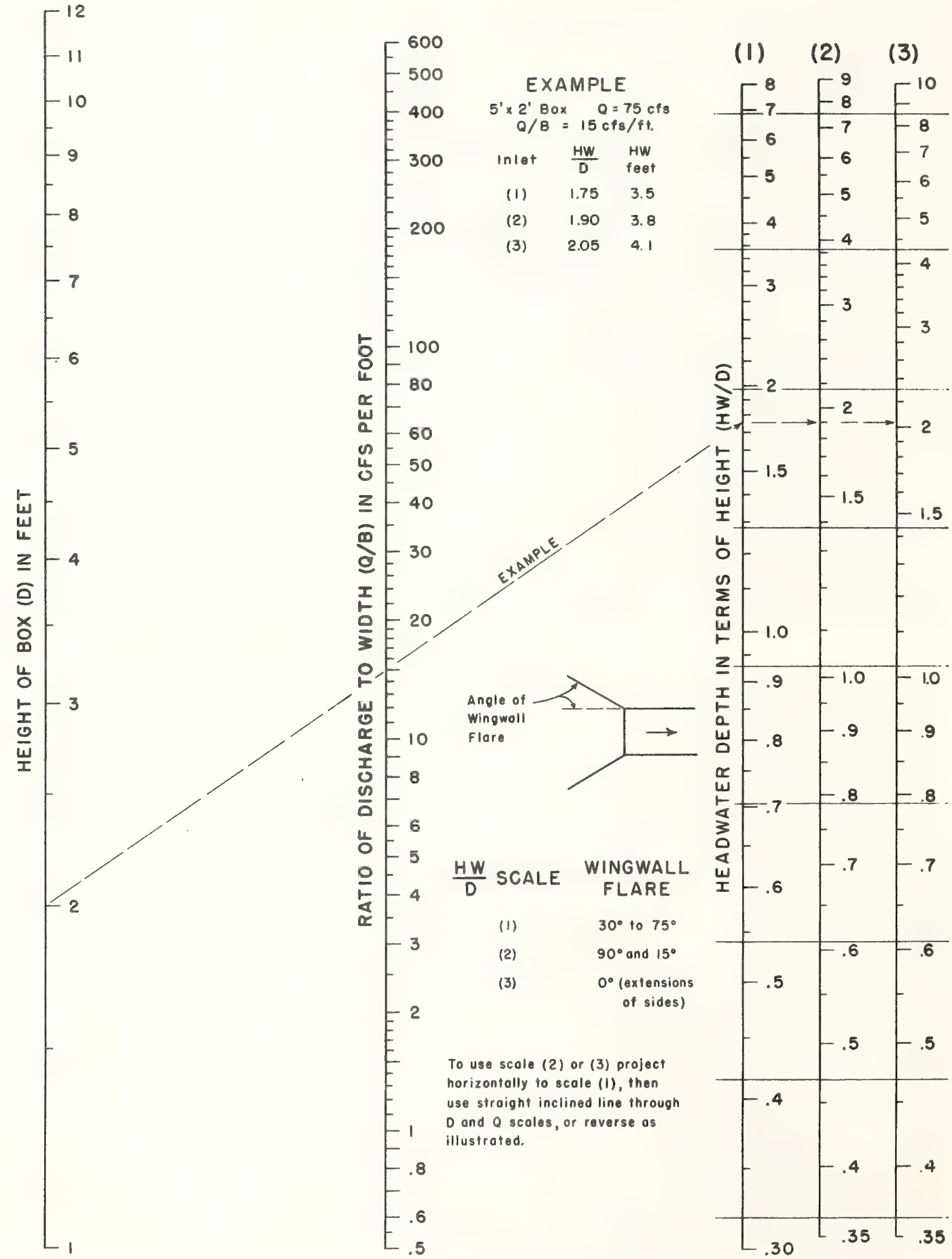
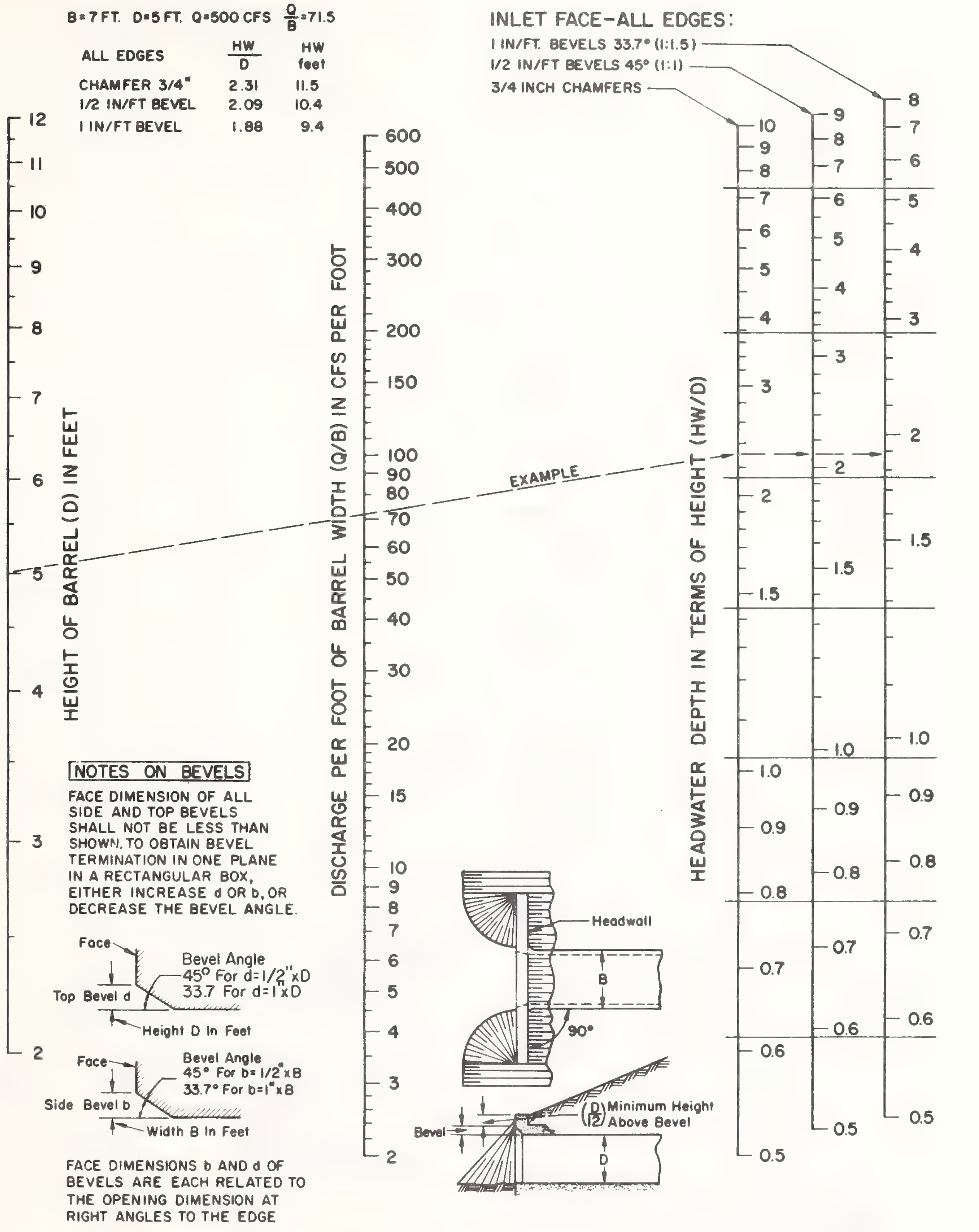
CRITICAL DEPTH
STRUCTURAL PLATE
C. M. PIPE-ARCH
18 INCH CORNER RADIUS



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

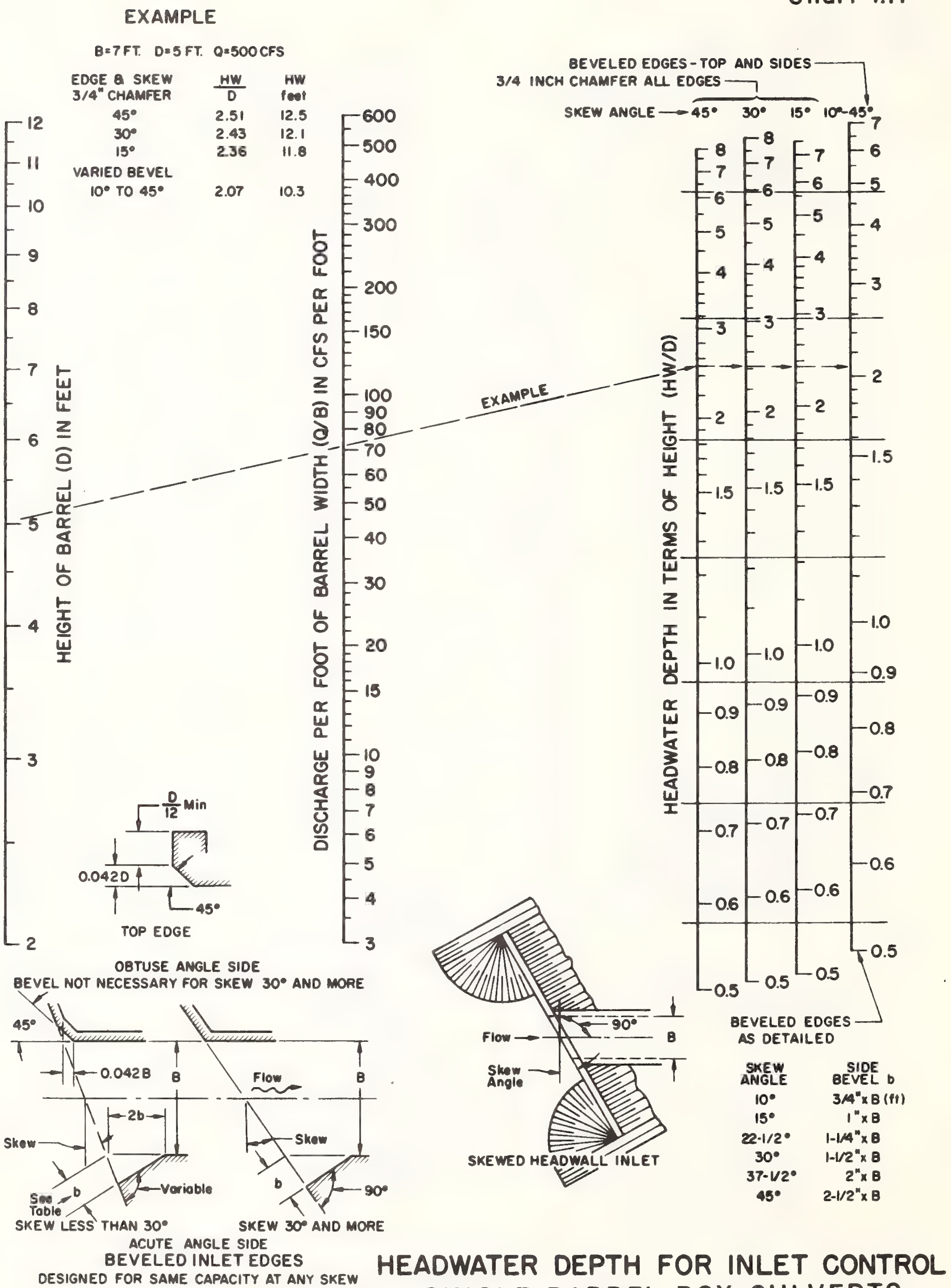
Chart 4.16



HEADWATER DEPTH FOR INLET CONTROL RECTANGULAR BOX CULVERTS 90° HEADWALL CHAMFERED OR BEVELED INLET EDGES

FEDERAL HIGHWAY ADMINISTRATION
AUGUST 1968

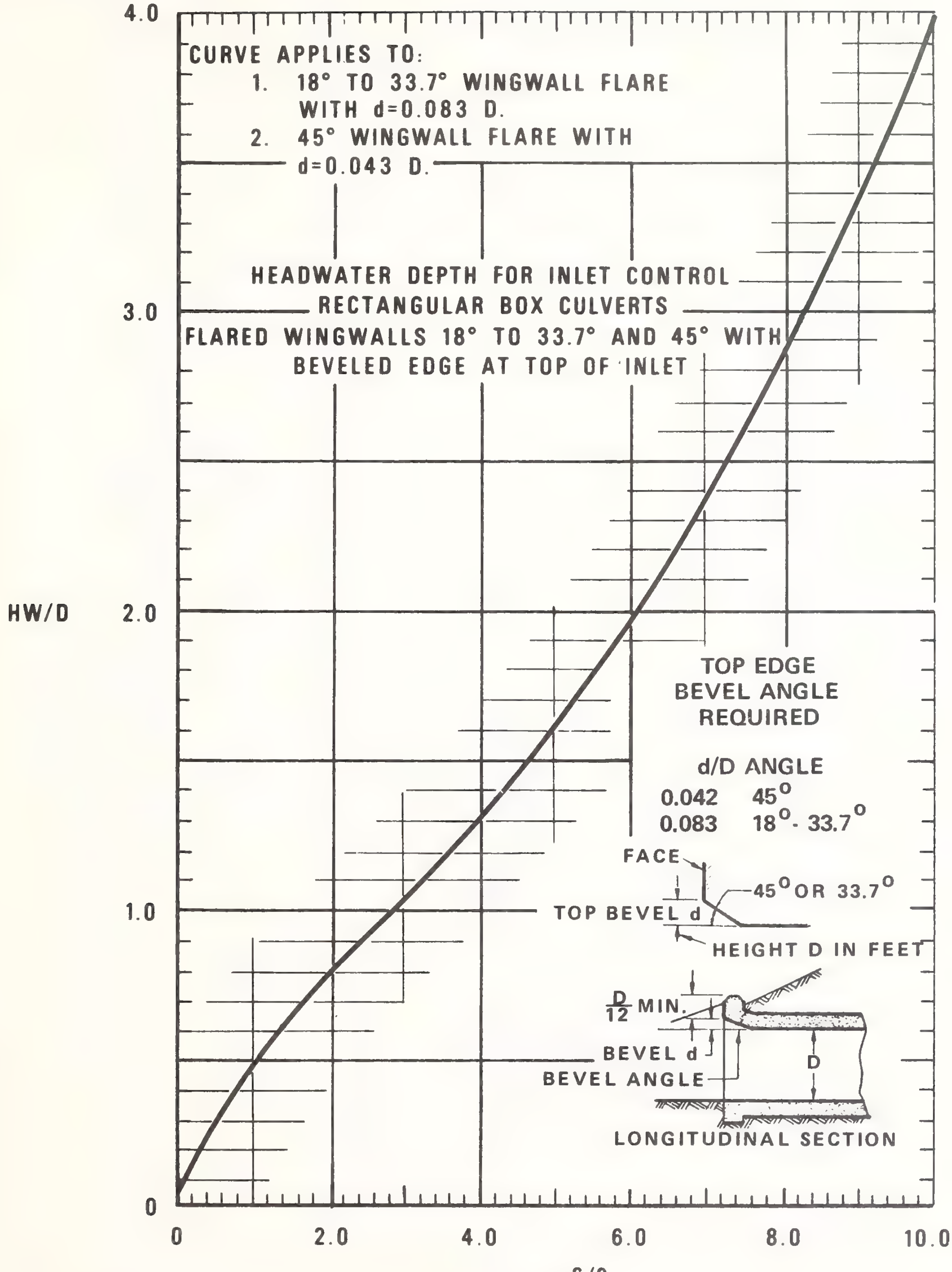
Chart 4.17



HEADWATER DEPTH FOR INLET CONTROL
SINGLE BARREL BOX CULVERTS
SKEWED HEADWALLS
CHAMFERED OR BEVELED INLET EDGES

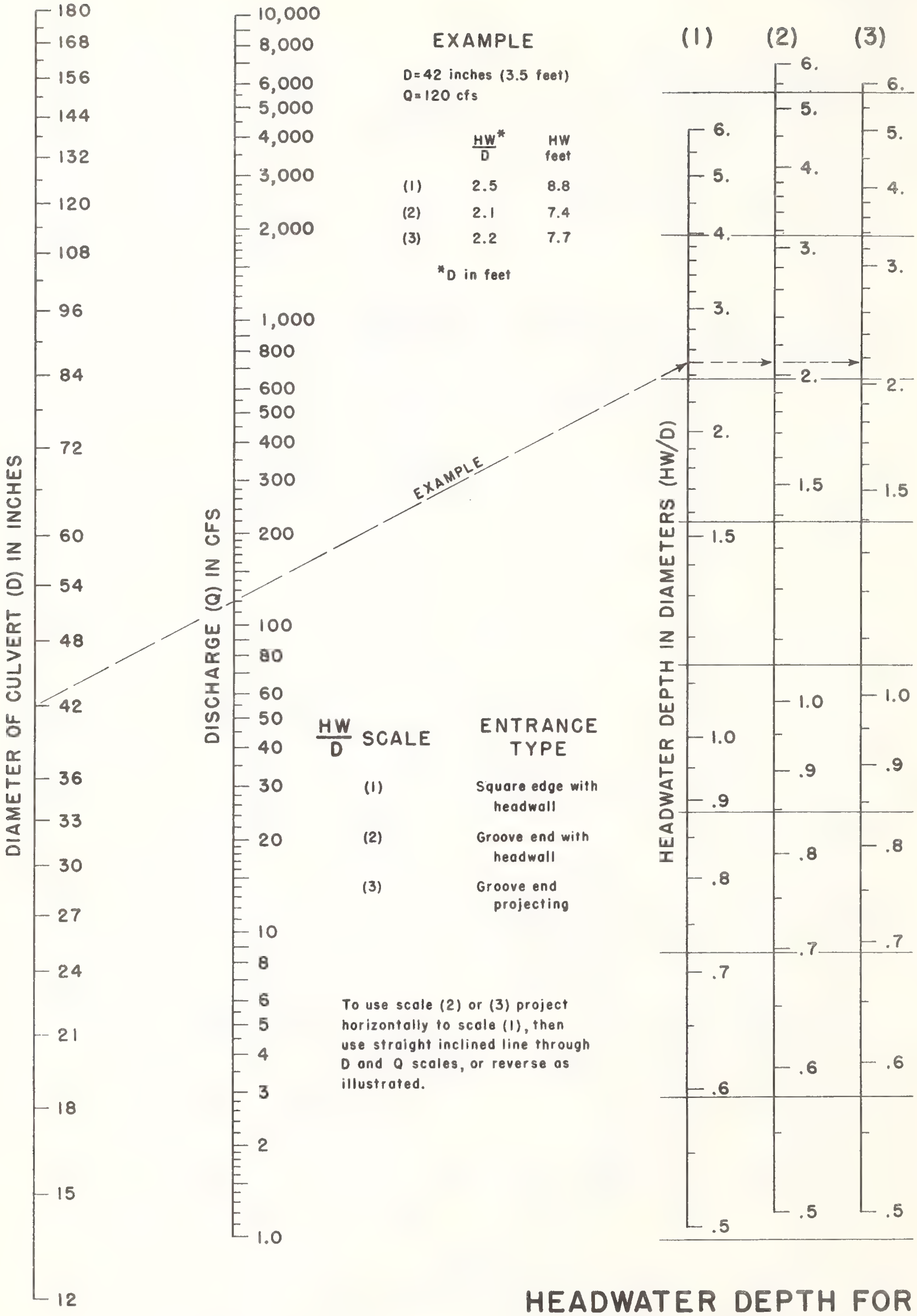
FEDERAL HIGHWAY ADMINISTRATION
AUGUST 1968

Chart 4.18



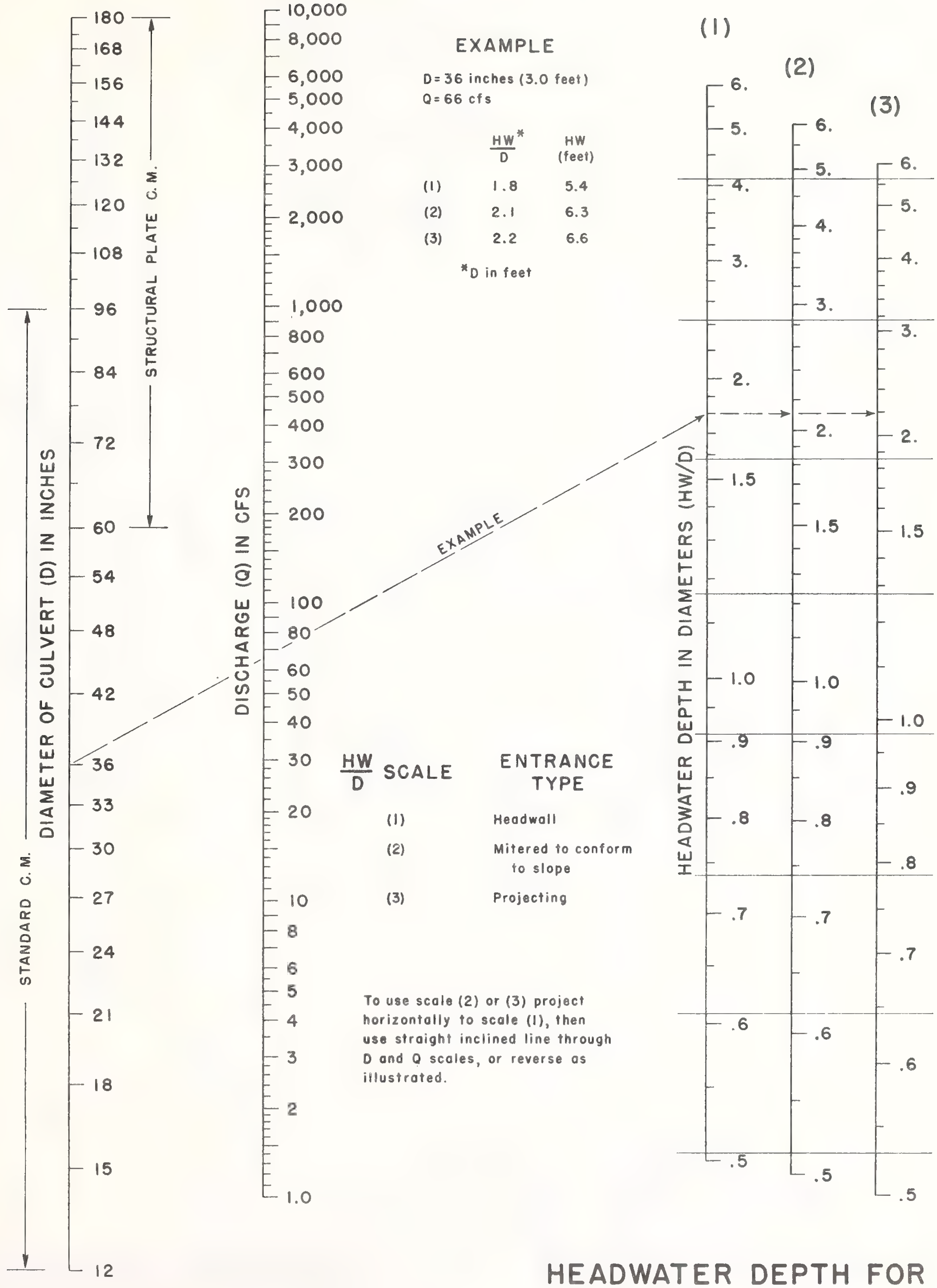
FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971

HEADWATER DEPTH FOR INLET CONTROL
RECTANGULAR BOX CULVERTS
FLARED WINGWALLS 18° TO 33.7° AND 45°
WITH BEVELED EDGE AT TOP OF INLET



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

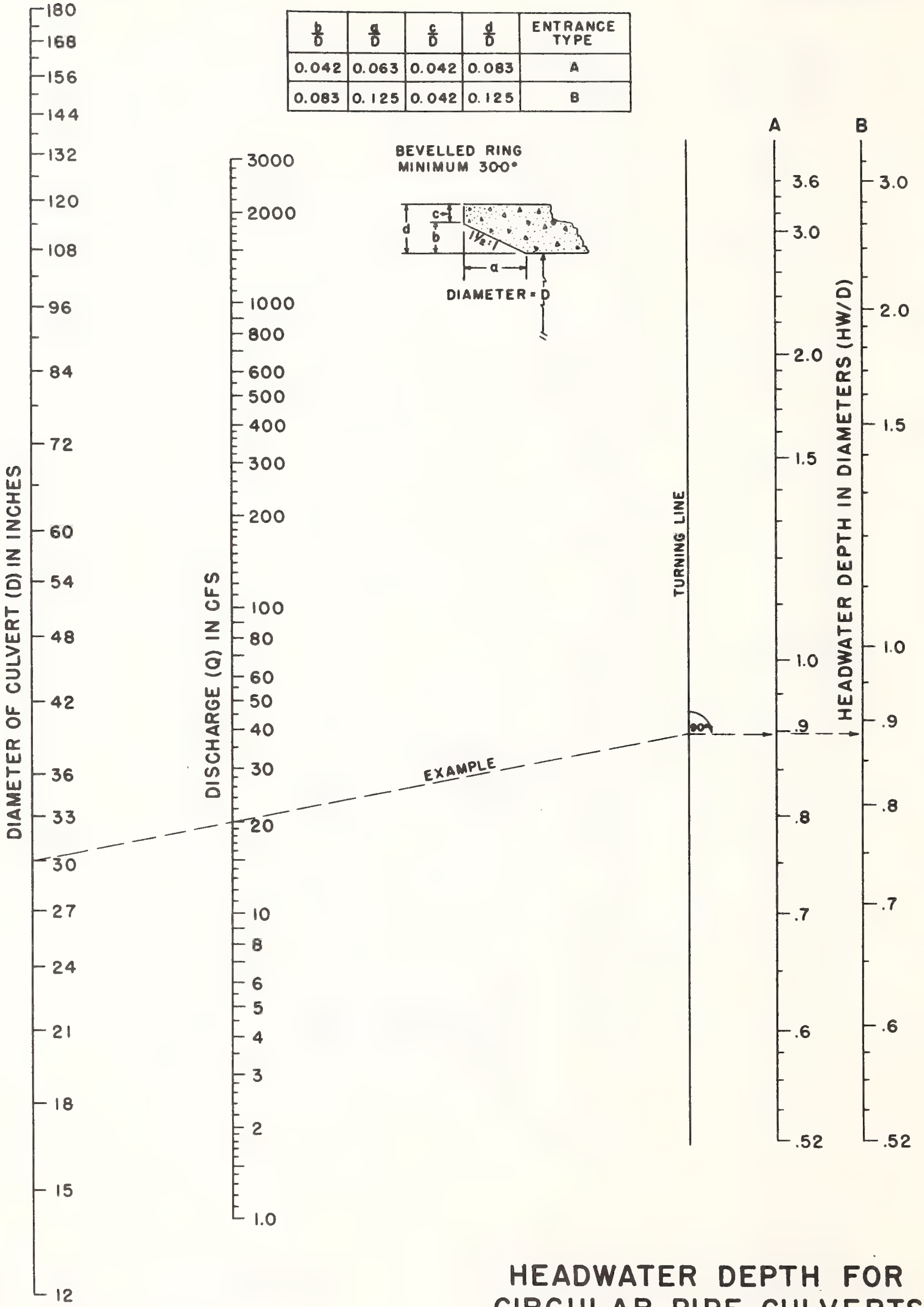
BUREAU OF PUBLIC ROADS JAN. 1963



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

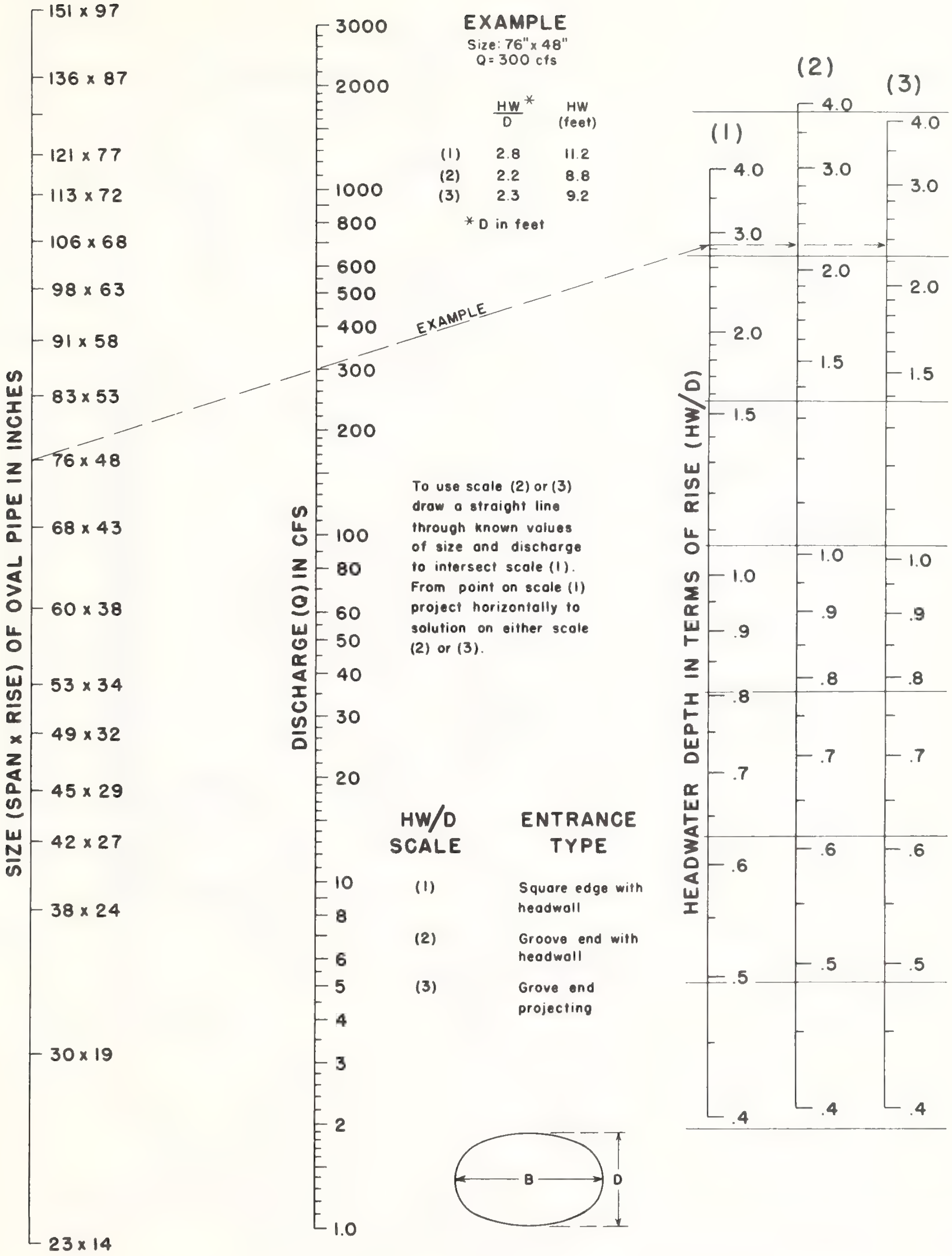
BUREAU OF PUBLIC ROADS JAN. 1963

b/D	a/D	c/D	d/D	ENTRANCE TYPE
0.042	0.063	0.042	0.083	A
0.083	0.125	0.042	0.125	B



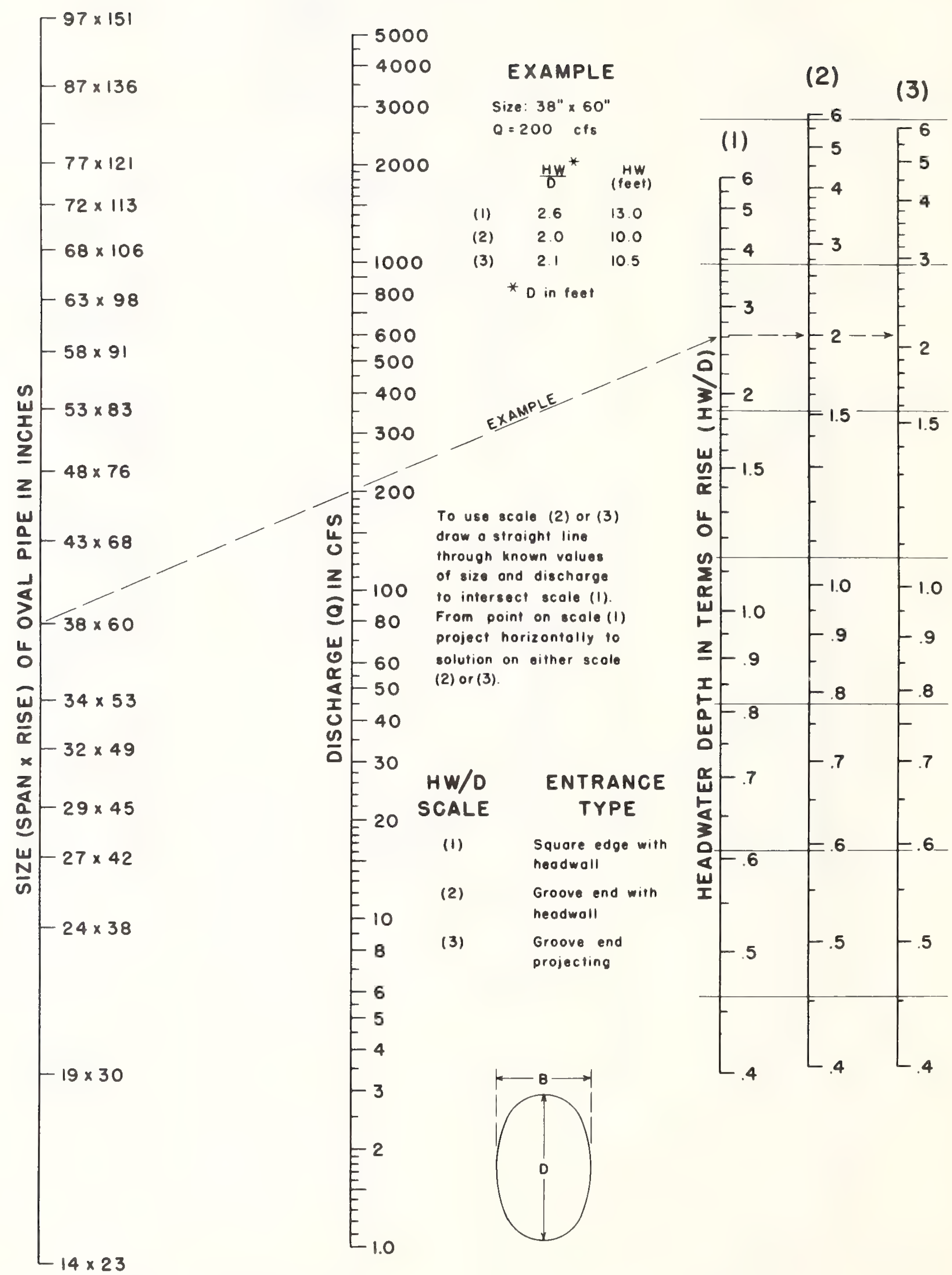
BUREAU OF PUBLIC ROADS
MARCH 1964

HEADWATER DEPTH FOR CIRCULAR PIPE CULVERTS WITH BEVELED RING INLET CONTROL



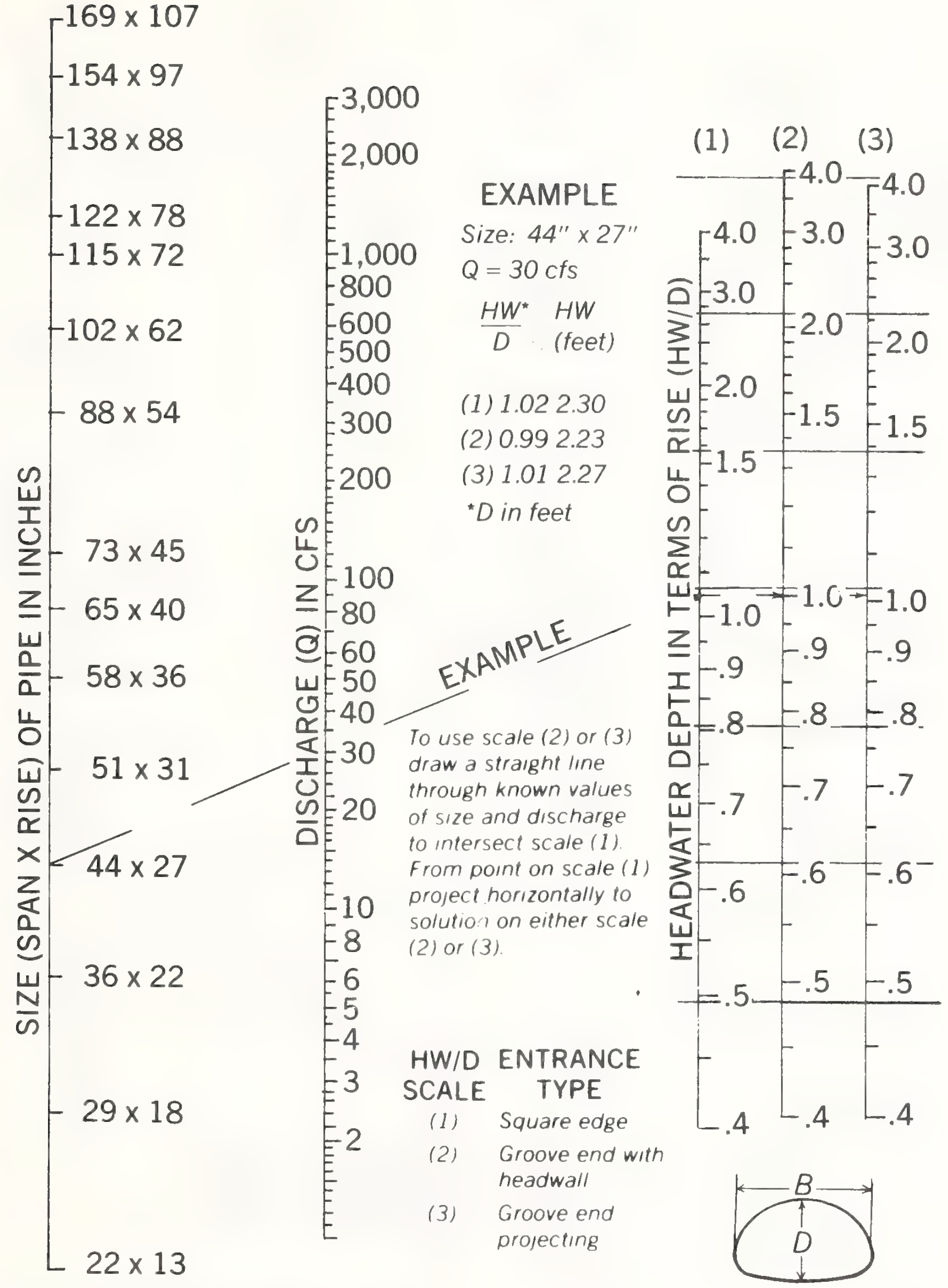
HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS HORIZONTAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



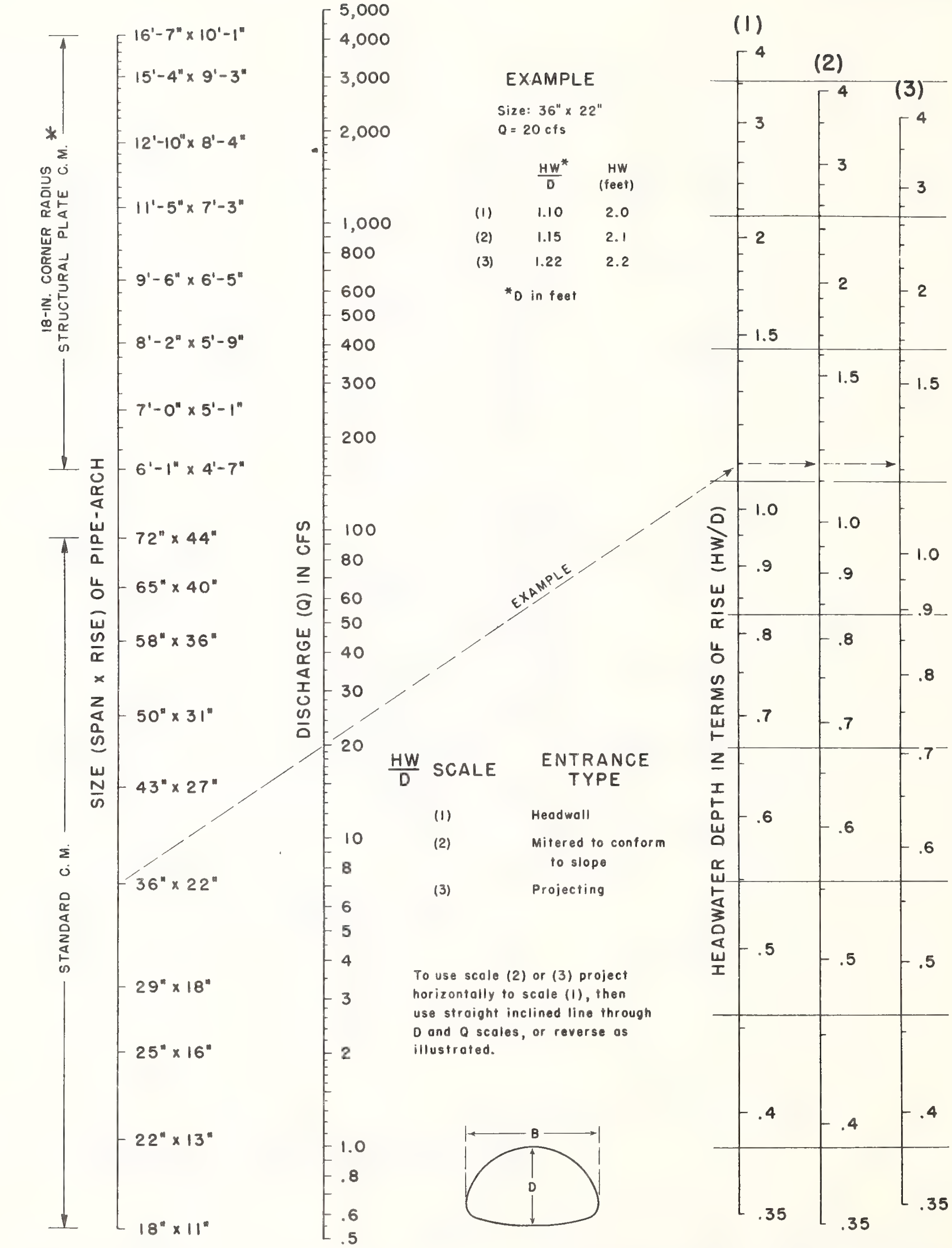
HEADWATER DEPTH FOR
OVAL CONCRETE PIPE CULVERTS
LONG AXIS VERTICAL
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963



American Concrete Pipe Association 1970

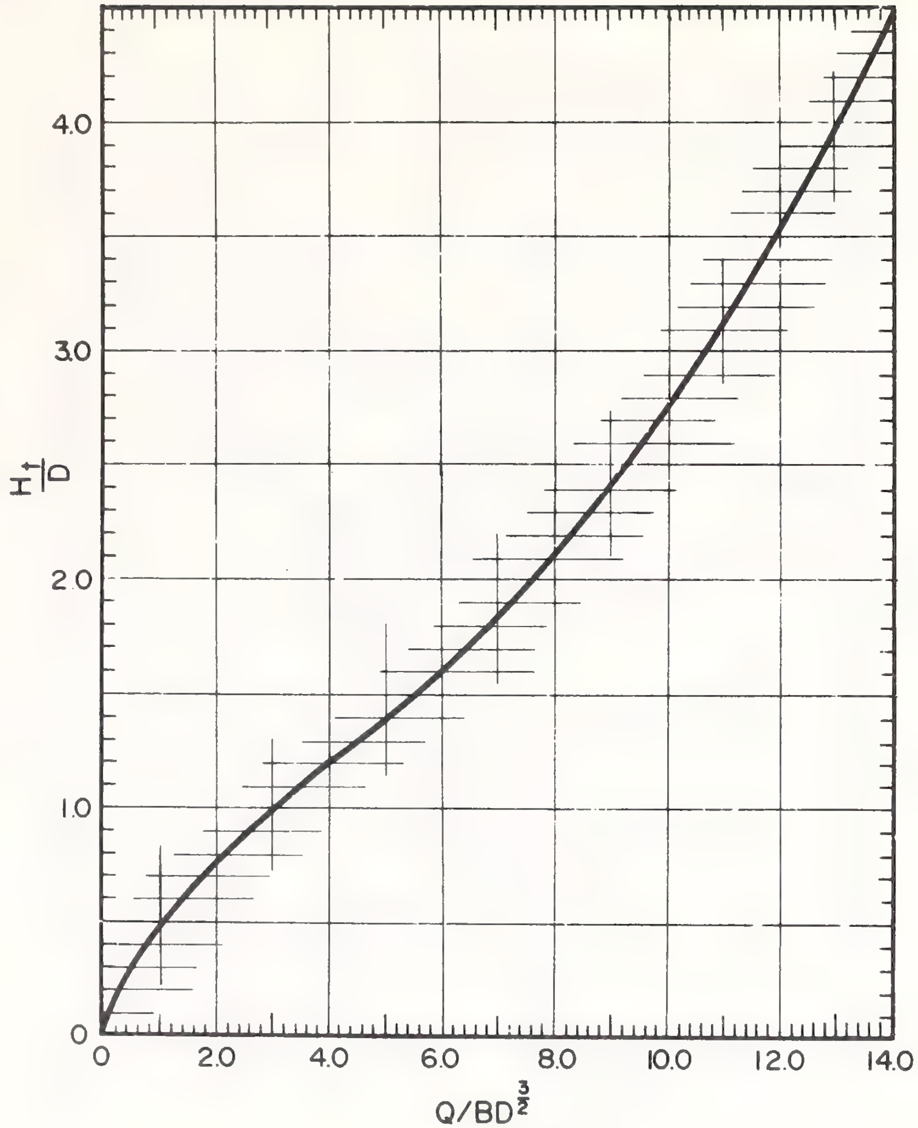
HEADWATER DEPTH FOR CONCRETE ARCH CULVERTS WITH INLET CONTROL



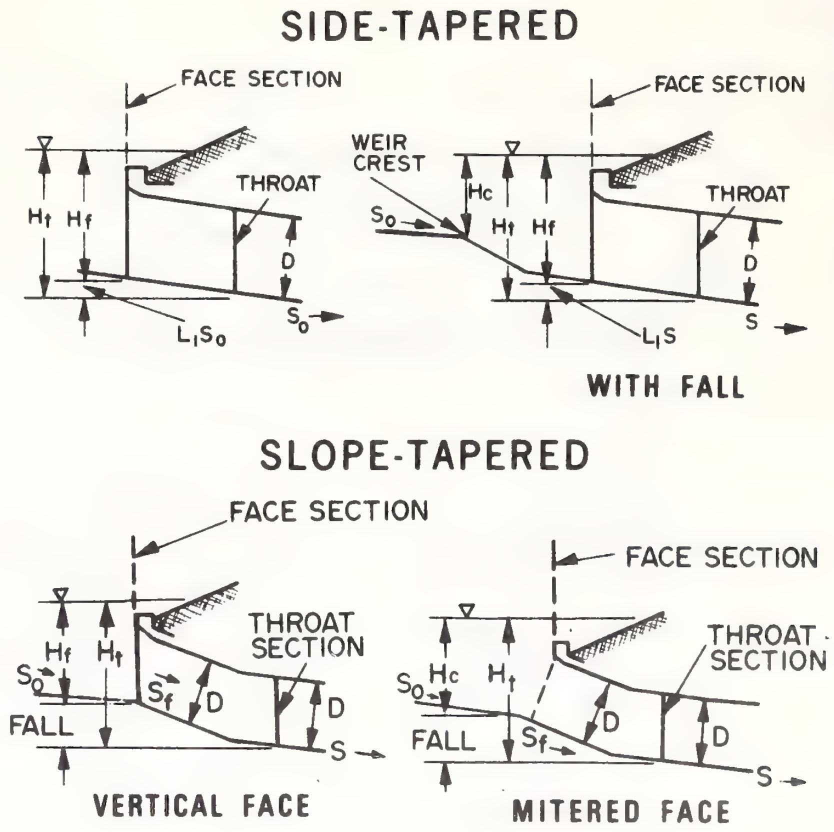
*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

BUREAU OF PUBLIC ROADS JAN. 1963

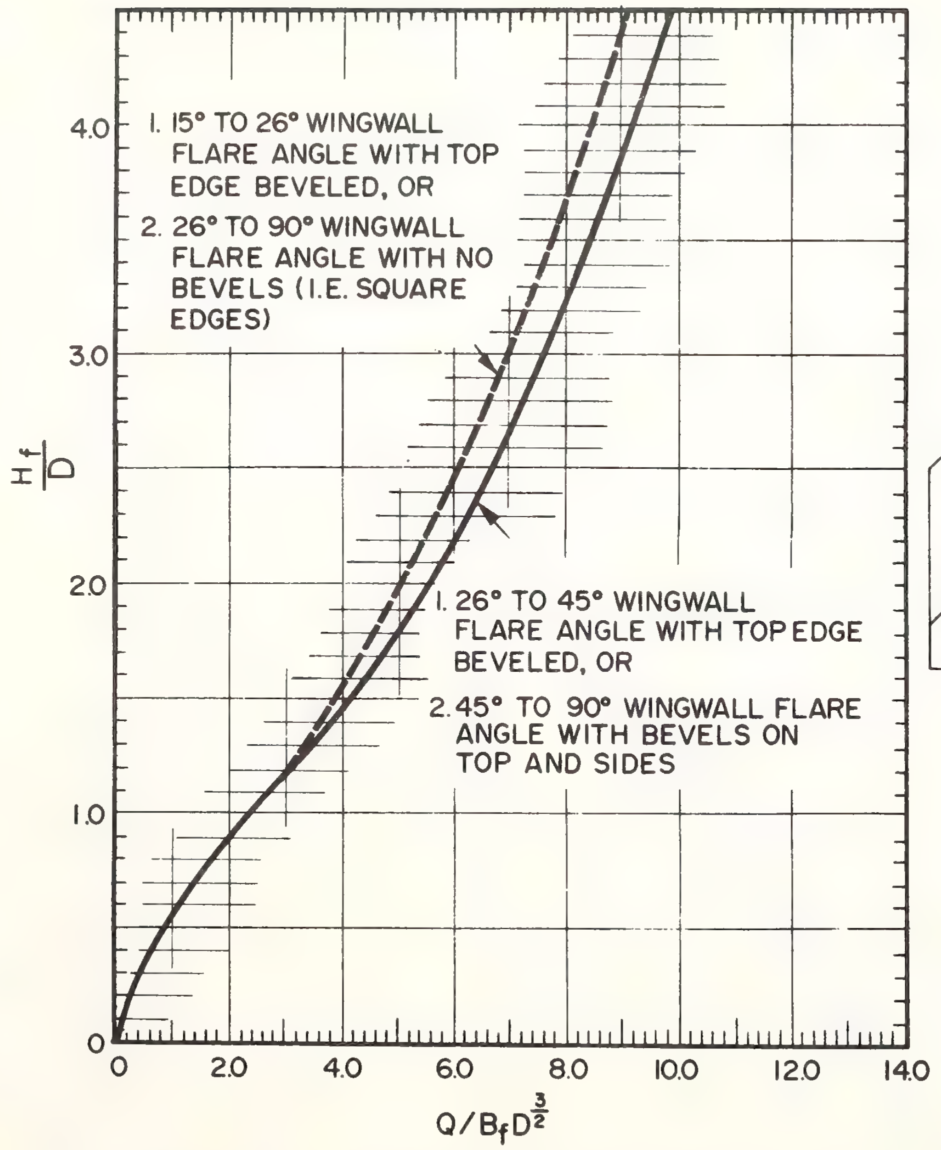
HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

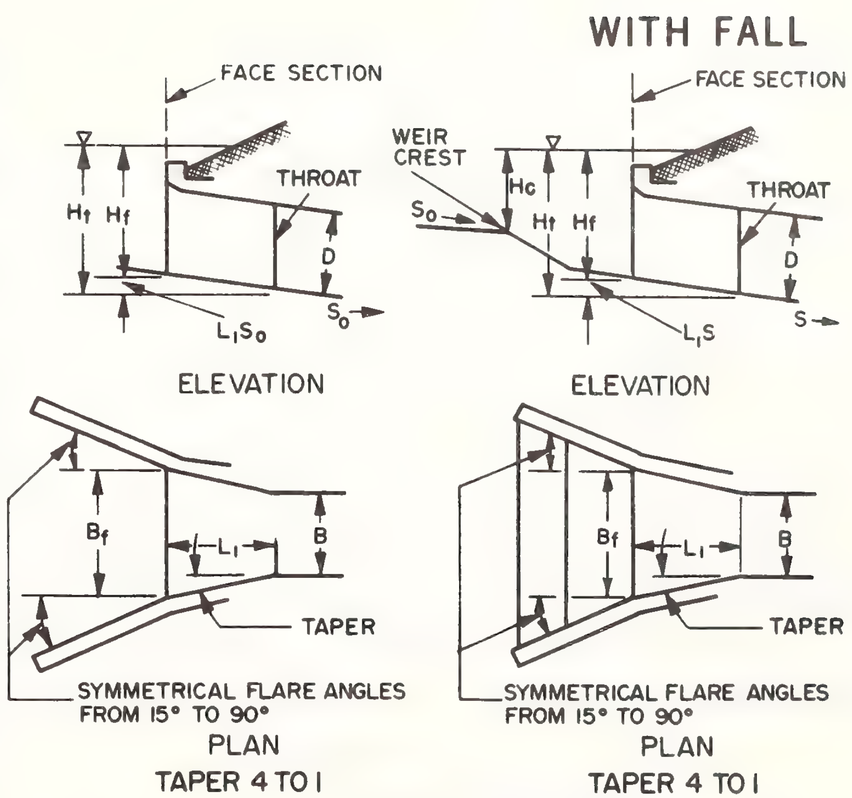


FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971



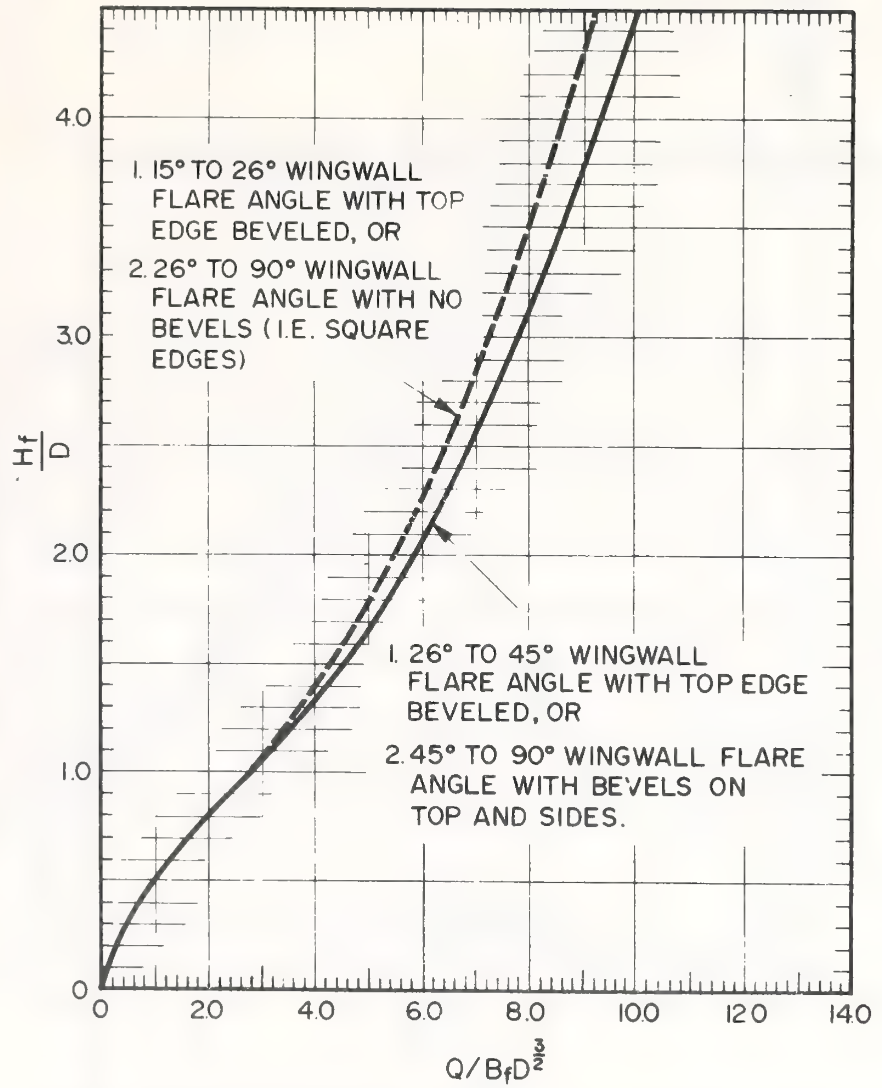
THROAT CONTROL CURVE FOR BOX CULVERTS TAPERED INLETS



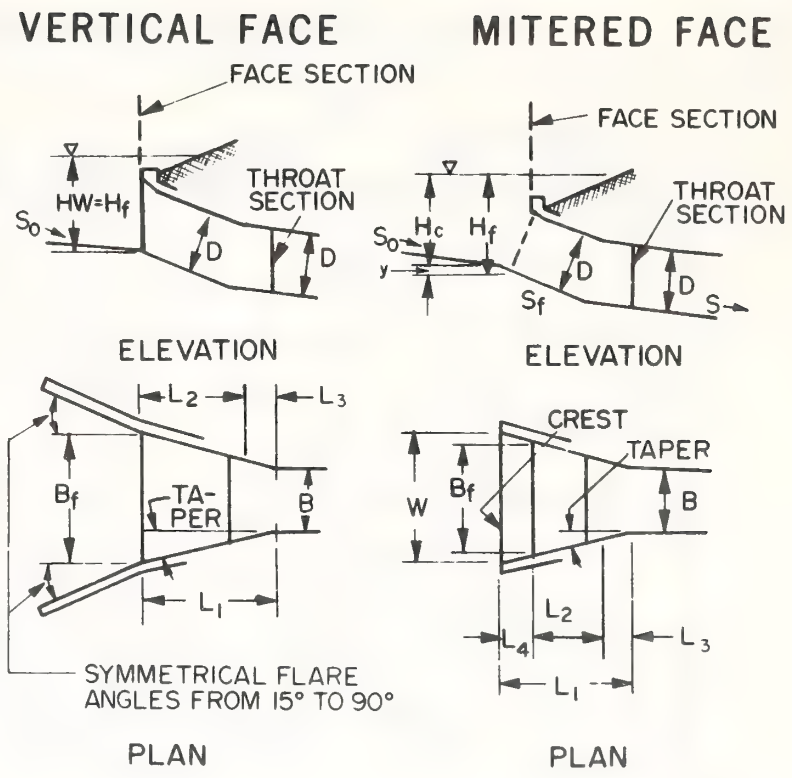


FACE CONTROL CURVES FOR BOX CULVERTS SIDE-TAPERED INLETS

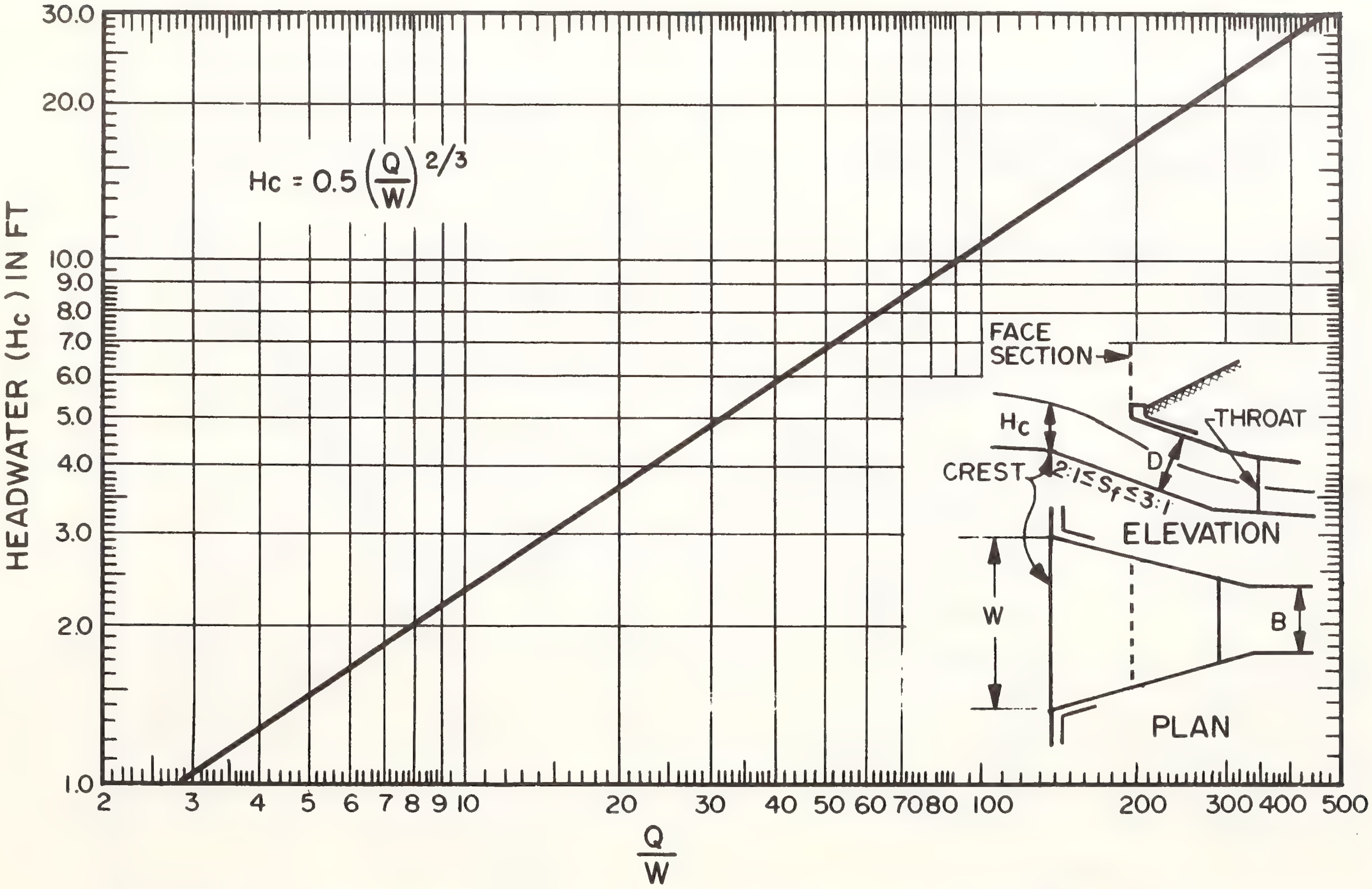
FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971



FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971

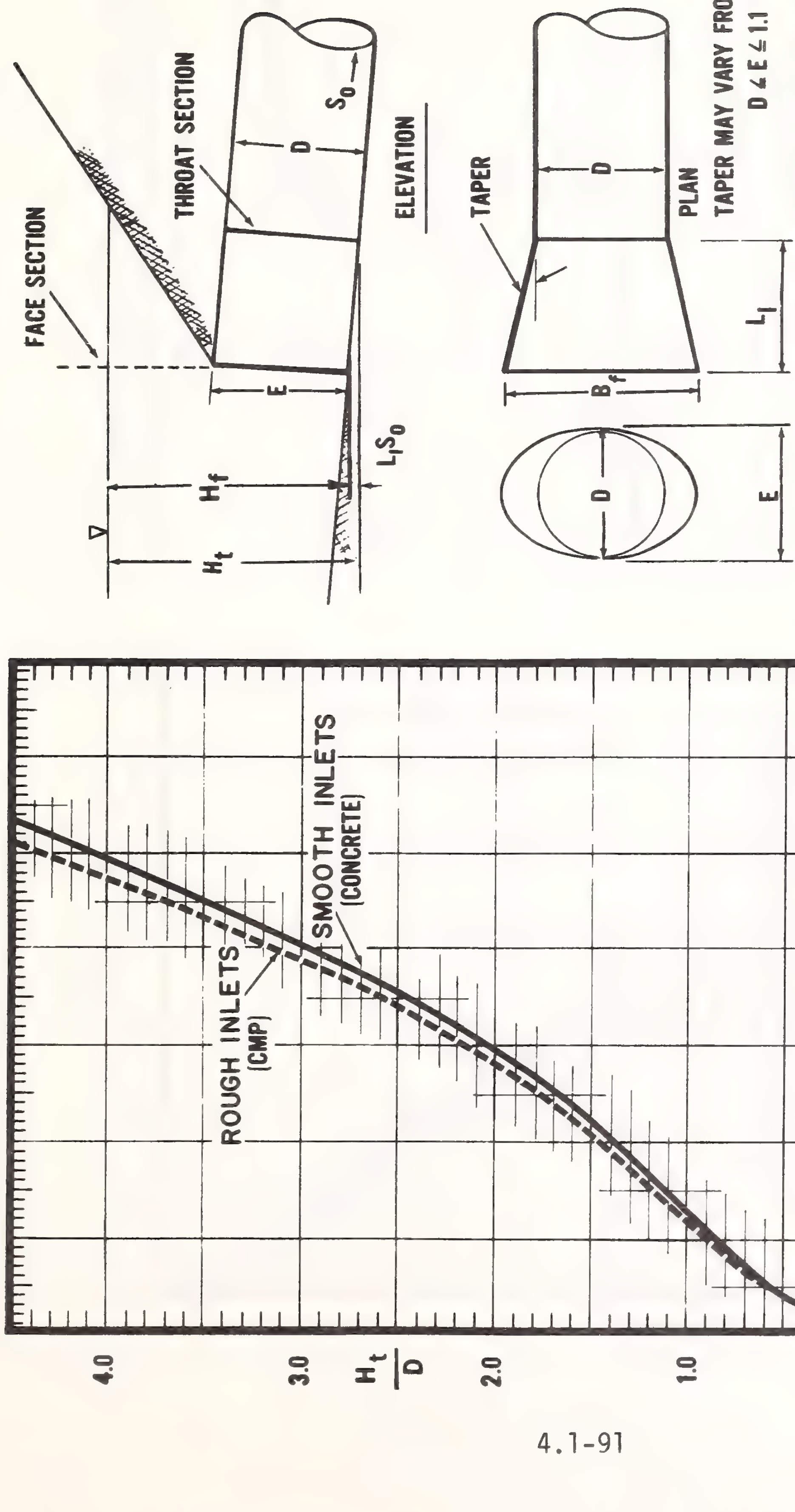


FACE CONTROL CURVES FOR BOX CULVERTS SLOPE-TAPERED INLETS



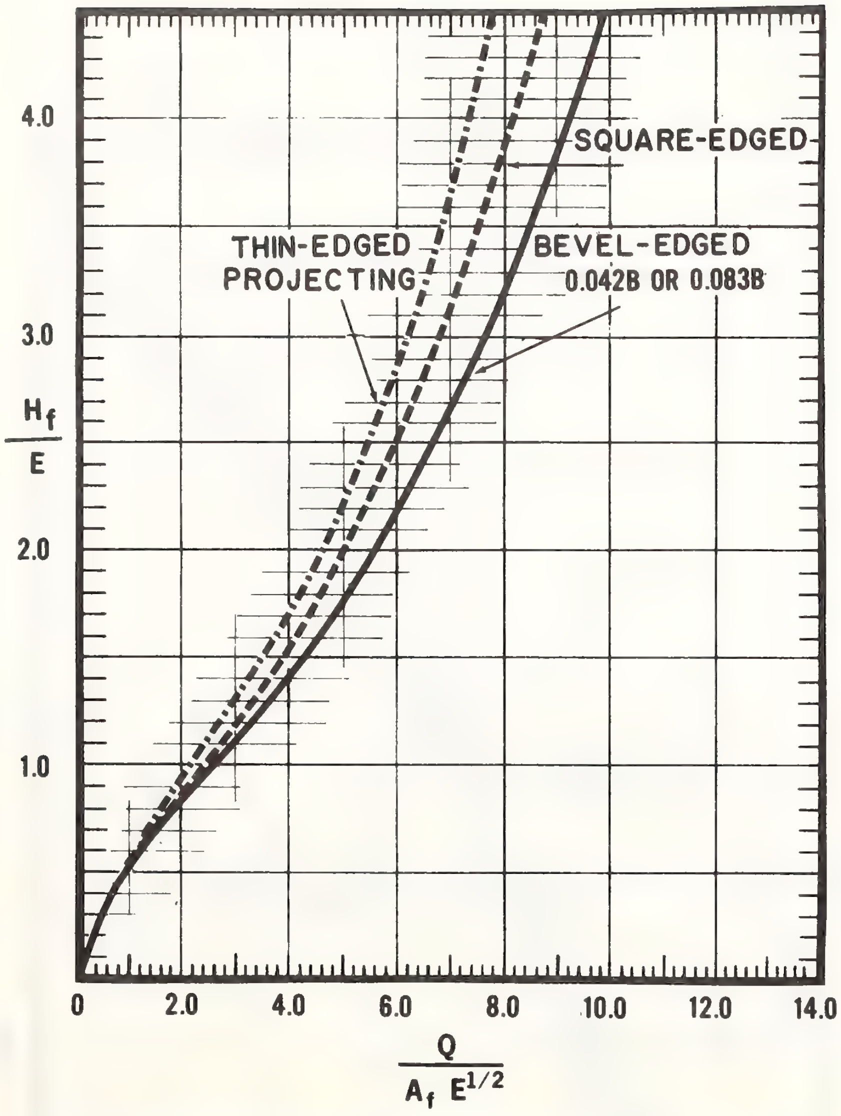
HEADWATER REQUIRED FOR CREST CONTROL

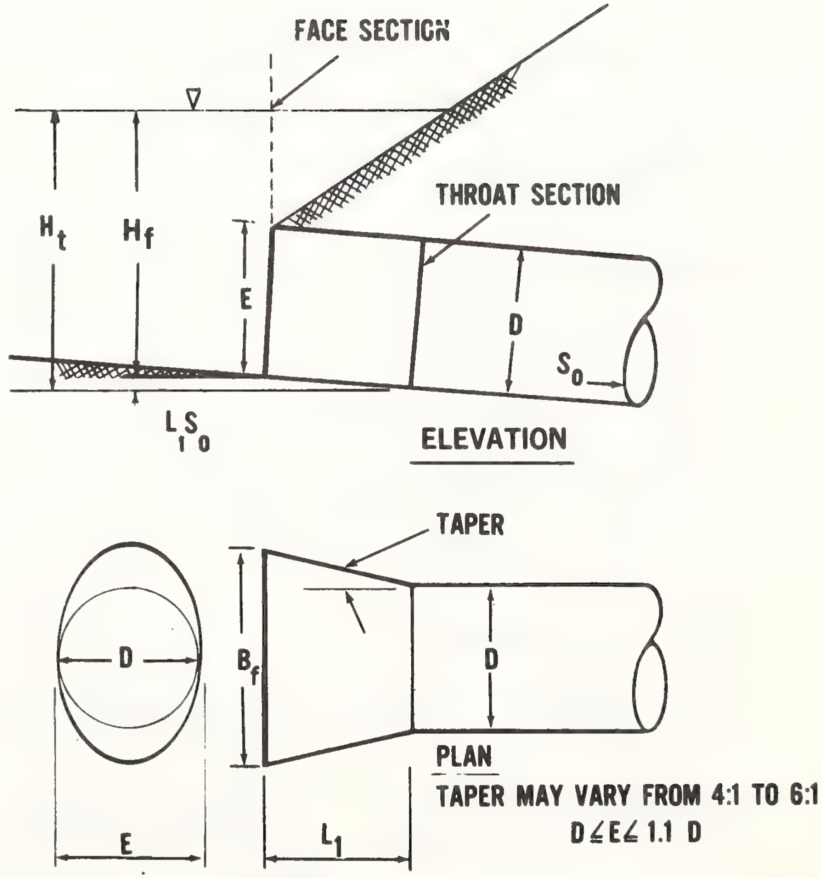
FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971



THROAT CONTROL CURVES
FOR SIDE-TAPERED INLETS TO PIPE CULVERT
(CIRCULAR SECTIONS ONLY)

FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971





FACE CONTROL CURVES
FOR SIDE- TAPERED INLETS TO PIPE CULVERTS
(NON-RECTANGULAR SECTIONS ONLY)

FEDERAL HIGHWAY ADMINISTRATION
OCTOBER 1971

EXAMPLE PROBLEMS

BOX CULVERT EXAMPLE NO. 1

Given: Design Discharge (Q) - 1,000 cfs, for a 50-year recurrence interval

Slope of stream bed (S_o) = 0.05 ft./ft.

Allowable Headwater Elevation = 200

Elevation Outlet Invert - 172.5

Culvert Length (L_a) = 350 ft.

Downstream channel approximates an 8' wide trapezoidal channel with 2:1 side slopes and a Manning's "n" of 0.03.

Requirements: This box culvert will be located in a rural area where the Allowable Headwater Elevation is not too critical; that is, the damages are low due to exceeding that elevation at infrequent times. Thus, the culvert should have the smallest possible barrel to pass design Q without exceeding AHW El. Use a reinforced concrete box with n = 0.012.

PROJECT: Example No 1 | OUTLET CONTROL DESIGN CALCULATIONS | DESIGNER: ______

STATION: ______ | | DATE: ______

INITIAL DATA:

Q_{50} = 1000 cfs

AHW El. = 200 ft.

S_o = 0.05

L_a = 350 ft.

El. Outlet Invert 172.5 ft.

Stream Data:

n = 0.03

S_o = 0.05

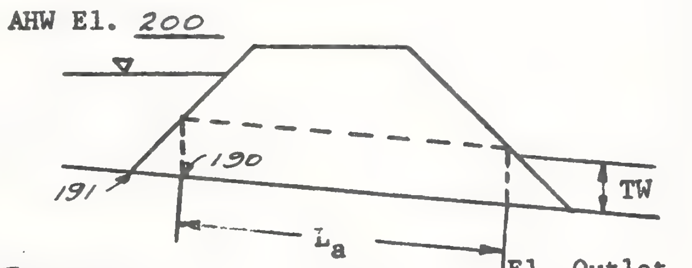
1

2

8'

Barrel Shape and Material Rect. Conc. Box

SKETCH



First Cut

Q = 1000; k_e = 0.5; L_a = 350

H = 200 - 172.5 - 5 = 22.5

∴ A = 40 ft² ∴ Try 7'x6'

Q	Q/N	H *	Q/NB	(1) d_c	$\frac{d_c+D}{2}$	Q_n	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. 1, N = 1, B = 7, D = 6, k_e = 0.5 Sq. Edges											
1000	1000	21.	143.	>6	6.0		3.5	6.0	199.5	23.8	OK - Close to AHW El.
800	800	13.2	114.	>6	6.0		3.25	6.0	191.7		
1200	1200	30.	172.	>6	6.0		3.8	6.0	208.5		
Trial No. 2, N = 1, B = 7, D = 6, k_e = 0.2 Beveled Edges											
1000	1000	19.	143	>6	6.0		3.5	6.0	197.5	23.8	OK - Lowered HW_o 2'
800	800	12.		Same	as			6.0	190.5		Try 1 - 6'x6'
1200	1200	27.		Sq Edge				6.0	205.5		
Trial No. 3, N = 1, B = 6, D = 6, k_e = 0.2 Beveled Edges											
1000	1000	26.	167	>6.0	6.0		3.5	6.0	204.5	27.8	No Good - Does Not Meet
											Design Criteria
											Exceed AHW El.

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.

(4) HW_o = H + h_o + El. Outlet Invert

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D.

SELECTED DESIGN

N = 1

B = 7 ft.

D = 6 ft.

k_e = 0.2

At Design Q:

HW_o = 197.5 ft.

V_o = 23.8 f/s

$$* \quad H = \left[1 + k_e + \frac{29 n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

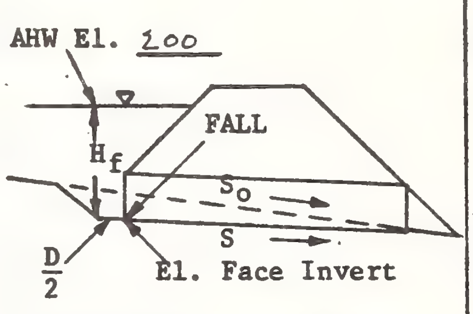
** Number of Barrels

PROJECT: EXAMPLE No. 1 STATION: ______	CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS	DESIGNER: ______ DATE: ______

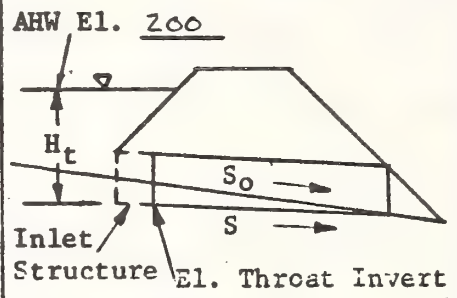
INITIAL DATA:

Q_{50} = 1000 cfs
AHW El. = 200 ft.
S_o = 0.05
L_a = 350 ft.
El. Stream Bed at Face 190 ft.

Barrel Shape and Material RCB; n=0.012
N = 1, B = 7
D = 6, $NBD^{3/2}$ = 102.9



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) / FALL	(3) HW_f / HW_t	(4) / S	(5) / V_o	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat / COMMENTS
Trial No. 1				Inlet and Edge Description BEVEL-EDGED INLET						
1000	143	3.9	23.4	176.6	192*	15.4	200			FALL TOO LARGE, TRY TAPERED
*ADJUSTED UPSTREAM DUE TO FALL						*				INLET - DO NOT USE BEVEL-
										EDGED INLET
Trial No. 2				Inlet and Edge Description TAPERED INLET THROAT						
1000	9.72	2.65	15.9	184.1	190	5.9	200.	0.033	34.2	OK- CALC. PERFORMANCE CURVES.
800	7.79	2.05	12.3	X	X	X	196.4			FROM PLOT, OPPORTUNITY TO GAIN (FALL = 1.3 + 5.9 = 7.2')
1200	11.68	3.4	20.4				204.5			MAX. CAPACITY AT AHW = 200.
Trial No. 3				Inlet and Edge Description TAPERED INLET THROAT FALL = 7.2'						
1000			15.9	182.8	190	7.2	198.7	0.029	33.3	OK - CAPACITY AT AHW=200
800			12.3			X	195.1			1062 cfs.
1200			20.4				203.2			

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)
(2) FALL=El. Stream Bed at Face−El. face (or throat) invert
(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed
(4) $S \approx S_o$ − FALL/L_a
(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:
FALL = 7.2 ft.
Invert El. = 182.8 ft.
Bevels:
Angle = ______
b = ______ in., d = ___ in.

PROJECT: Example No. 1

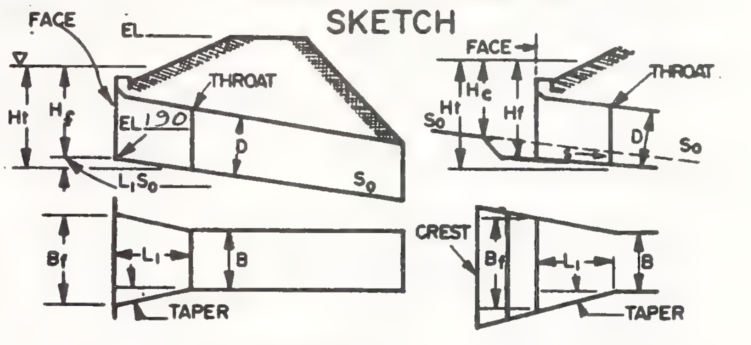
STATION: ______

SIDE-TAPERED INLET
DESIGN CALCULATIONS

DESIGNER: ______
DATE: ______

INITIAL DATA

Q_{50} = 1000 cfs S_o = 0.05
AHW El. = 200 ft. L_a = 350 ft.
TAPER = 4 :1
Barrel Shape and Material RCB ; n=0.012
Face Edge Description 45° Bevels



N = 1 , B = 7 , D = 6

Q	El. Throat Invert	(1) $\frac{H_f}{D}$ ~~$\frac{H_f}{E}$~~	$\frac{Q}{B_f D^{3/2}}$ ~~$\frac{Q}{A_f E^{1/2}}$~~	$D^{3/2}$ ~~$E^{1/2}$~~	(2) Min. B_f ~~Min. A_f~~	B_f	(3) L_1	(4) S	L_1S	(5) El. Face Invert	COMMENTS
Trial No. 1 , Q = 1000 , HW_f = 200 (Min. Req'd) Fall = 5.9'											
1000	184.1	2.48	6.6	14.7	10.3	10.5	7.0	0.033	0.2	184.3	$B_fD^{3/2}$ [or $A_fE^{1/2}$] = 154
											Q · $Q/B_fD^{3/2}$ · H_f/D · H_f · HW_f
											900 · 5.84 · 2.14 · 12.8 · 196.9
											1000 · 6.50 · 2.42 · 14.5 · 198.6
											1100 · 7.14 · 2.77 · 16.6 · 200.7
Trial No. 2 , Q = 1000 , HW_f = 198.7 (Fall = 7.2')											
1000	182.8	2.48	6.6	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_fD^{3/2}$ [or $A_fE^{1/2}$] = ______
Trial No. 3 , Q = 1062 , HW_f = 200 (Fall = 7.2)											
1062	182.8	2.70	7.05	14.7	10.3	10.5	7.0	0.029	0.2	183.0	$B_fD^{3/2}$ [or $A_fE^{1/2}$] = ______

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1)/D [or E]
  $D \leq E \leq 1.1D$

(2) $\text{Min. } B_f = Q \Big/ \left[(D^{3/2}) \frac{Q}{B_f D^{3/2}} \right]$

  $\text{Min. } A_f = Q \Big/ \left[(E^{1/2}) \frac{Q}{A_f E^{1/2}} \right]$

(3) $L_1 = \left[\frac{B_f - NB}{2} \right]$ TAPER

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute. Face and Throat may be lowered to better fit site.

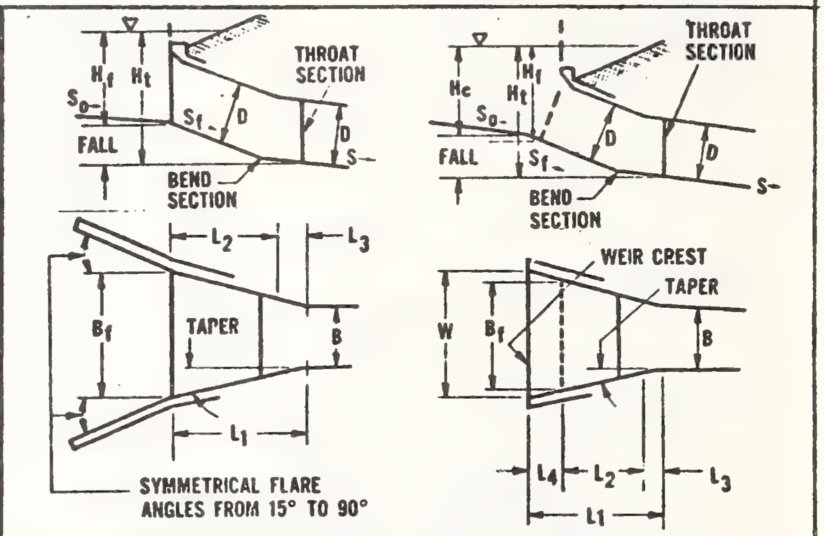
SELECTED DESIGN

B_f = 10.5 ft.
L_1 = 7.0 ft.
Bevels: Angle 45°
d = 3 in., b = 5.3 in.
Crest Check:
HW_c = 198.7 ft.
H_c = 7.7 ft.
Q/W = 59 (Chart 17)
Min. W = 17 ft.

PROJECT: Example No. 1	SLOPE-TAPERED INLET DESIGN CALCULATIONS	DESIGNER: ______
STATION: ______		DATE: ______

INITIAL DATA

Q= 1000 cfs S_o = 0.05
AHW El. = 200 ft. L_a = 350 ft.
El. Stream bed at crest 191 ft.
El. stream bed at face 190 ft.
TAPER = 4 :1 (4:1 to 6:1)
S_f = 2 :1 (2:1 to 3:1)
Barrel Shape and Material RCB ; n = 0.012
Inlet Edge Description 45° Bevels



VERTICAL = MITERED

N = 1 , B = 7 , D = 6

	Q	HW_f	El. Throat Invert	(1) El. Face Invert	(2) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	COMMENTS
Trial 1	1000	200	184.1	190	10	1.67	5.1	14.7	13.3	14	0.033	$B_f D^{3/2}$ = ______
												Vertical Face Point No. 1
Trial 2	1000	198.7	182.8	190	8.7	1.43	4.45	14.7	15.3	16	0.029	$B_f D^{3/2}$ = ______
												Vertical Face Point No. 2

Note: Use only throat designs with FALL > 0.25D
(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - 0.4D (Approx.), but higher than throat invert elevation.
(2) $H_f = HW_f$ - El. face invert
(3) Min $B_f = Q/[(D^{3/2})\ Q/B_f D^{3/2}]$

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj. L_3	(9) Adj. TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max. Crest El.	GEOMETRY
3.5	—	11.8	10.5	—	4.4:1	15.3					B_f = ___ft. L_3 = ___ ft. L_1 = ___ft. L_4 = ___ ft.
3.5	—	14.4	14.4	3.6	—	18.0					L_2 = ___ft. TAPER = ___:1

(4) Min L_3=0.5B
(5) $L_4 \approx S_f y + D/S_f$
(6) L_2=(El. Crest Invert-El. Throat Invert)$S_f - L_4$
(7) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER-L_3
(8) If (7)>(6), Adj.$L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER-L_2
(9) If (6)>(7) Adj.TAPER=$(L_2+L_3)/\left[\frac{B_f - NB}{2}\right]$
(10) $L_1 = L_2 + L_3 + L_4$
(11) Mitered: $W = NB + 2\left[\frac{L_1}{\text{TAPER}}\right]$
(12) Max. Crest El = $HW_f - H_c$

PROJECT: Example No. 1 STATION: ______	SLOPE-TAPERED INLET DESIGN CALCULATIONS	DESIGNER: ______ DATE: ______

INITIAL DATA

Q = 1060 cfs S_o = 0.05

AHW El. = 200 ft. L_a = 350 ft.

El. Stream bed at crest 191 ft.

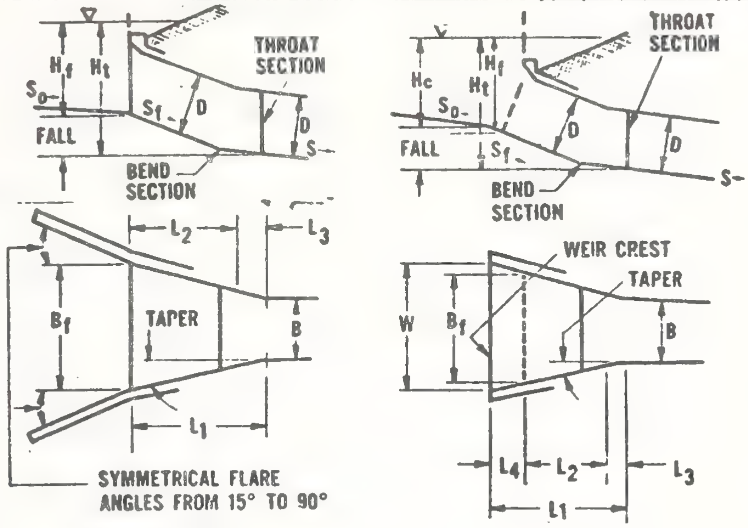
El. stream bed at face 190 ft.

TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material RCB, n=0.012

Inlet Edge Description 45° Bevels



VERTICAL = MITERED

N = 1 , B = 7 , D = 6

	Q	HW_f	El. Throat Invert	(1) El. Face Invert	(2) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	COMMENTS
Trial 1	1062	200	182.8	190	10	1.67	5.1	14.7	14.2	15.0	0.029	$B_f D^{3/2}$ = ______
												Vertical Face Point No. 3
Trial 2	1000	200	184.1	188.6	11.4	1.90	5.65	14.7	12.0	12.0	0.033	$B_f D^{3/2}$ = ______
												Mitered Face Point No. 1

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − 0.4D (Approx.), but higher than throat invert elevation.

(2) $H_f = HW_f$ − El. face invert

(3) Min $B_f = Q / [(D^{3/2})\ Q/B_f D^{3/2}]$

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj. L_3	(9) Adj. TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max. Crest El.	GEOMETRY
3.5	—	14.4	12.5	—	4.5:1	17.9					B_f = ___ft. L_3 = ___ ft. L_1 = ___ft. L_4 = ___ ft.
3.5	7.8	6.0	6.5	4.0	—	17.8	15.9	63.0	8.0	192.0	L_2 = ___ft. TAPER = ___:1

(4) Min $L_3 = 0.5B$

(5) $L_4 \approx S_f y + D/S_f$

(6) L_2 = (El. Crest Invert − El. Throat Invert) $S_f - L_4$

(7) Check $L_2 = \left[\frac{B_f - MB}{2}\right]$ TAPER − L_3

(8) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_2

(9) If (6)>(7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10) $L_1 = L_2 + L_3 + L_4$

(11) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12) Max. Crest El = $HW_f - H_c$

PROJECT: Example No. 1   **SLOPE-TAPERED INLET DESIGN CALCULATIONS**   DESIGNER: ____________

STATION: ____________   DATE: ____________

INITIAL DATA

Q=1000 cfs S_o = 0.05

AHW El. = 200 ft. L_a = 350 ft.

El. Stream bed at crest 191 ft.

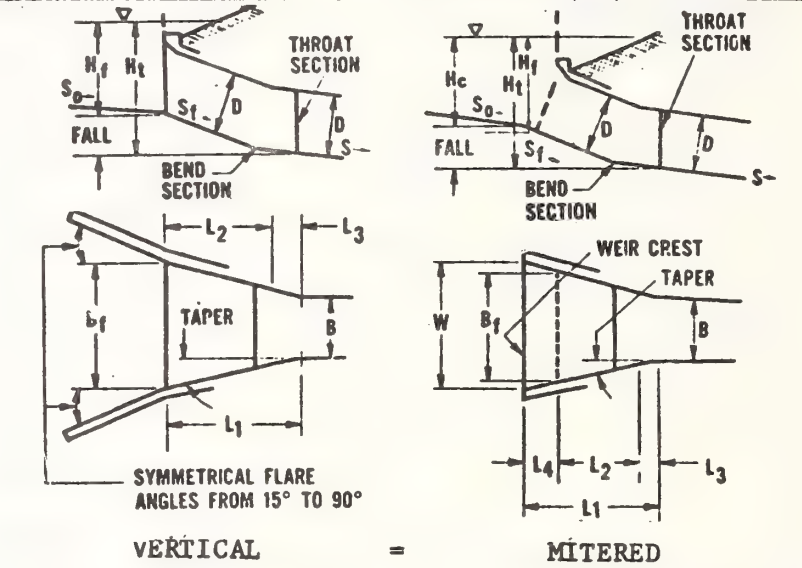
El. stream bed at face 190 ft.

TAPER = 4 :1 (4:1 to 6:1)

S_f = 2 :1 (2:1 to 3:1)

Barrel Shape and Material RCB ; n=0.012

Inlet Edge Description 45° Bevels



N = 1 , B = 7 , D = 6

	Q	HW_f	El. Throat Invert	(1) El. Face Invert	(2) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	COMMENTS
Trial 1	1000	198.7	182.8	188.6	10.1	1.68	5.15	14.7	13.5	14.0	0.029	$B_f D^{3/2}$ = ______
												Mitered Face Point No. 2
Trial 2	1062	200	182.8	188.6	11.4	1.90	5.65	14.7	12.8	13.0	0.029	$B_f D^{3/2}$ = ______
												Mitered Face Point No 3

Note: Use only throat designs with FALL > 0.25D

(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest - 0.4D (Approx.), but higher than throat invert elevation.

(2) $H_f = HW_f$ - El. face invert

(3) Min $B_f = Q/\left[(D^{3/2})\ Q/B_f D^{3/2}\right]$

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj. L_3	(9) Adj. TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max. Crest El.	GEOMETRY
											B_f = ___ft. L_3 = ___ ft.
											L_1 = ___ft. L_4 = ___ ft.
3.5	7.8	8.6	10.5	5.4	—	21.8	17.9	55.9	7.4	191.3	L_2 = ___ft.
3.5	7.8	8.6	8.5	—	4.03:1	19.9	16.8	63.3	7.9	192.1	TAPER = ___:1

(4) Min $L_3 = 0.5B$

(5) $L_4 = S_{fy} + D/S_f$

(6) L_2 = (El. Crest Invert - El. Throat Invert) $S_f - L_4$

(7) Check $L_2 = \left[\frac{B_f - MB}{2}\right]$ TAPER $- L_3$

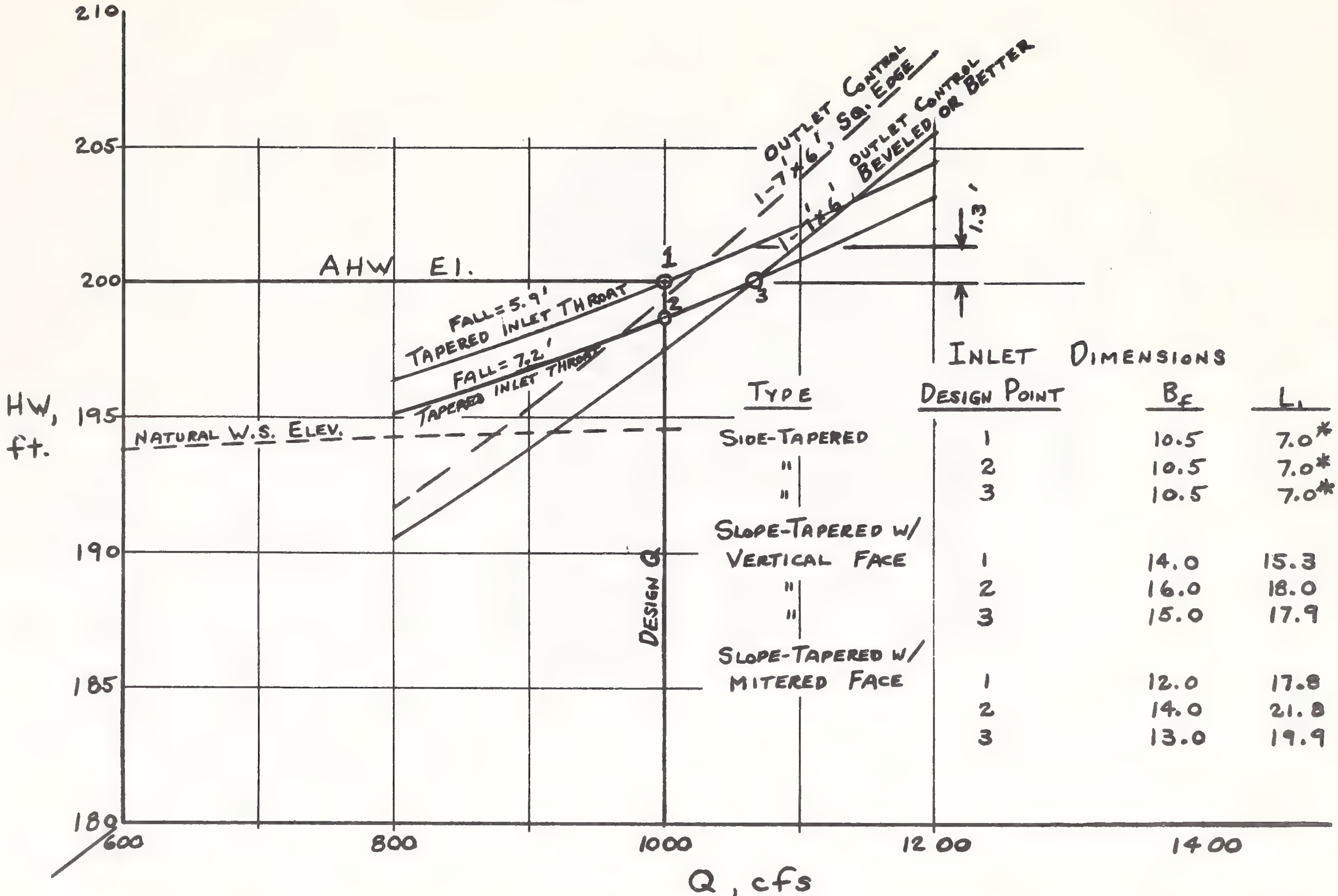
(8) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9) If (6)>(7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10) $L_1 = L_2 + L_3 + L_4$

(11) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12) Max. Crest El $= HW_f - H_c$



Inlet Dimensions

Type	Design Point	B_f	L_1
Side-Tapered	1	10.5	7.0*
"	2	10.5	7.0*
"	3	10.5	7.0*
Slope-Tapered w/ Vertical Face	1	14.0	15.3
"	2	16.0	18.0
"	3	15.0	17.9
Slope-Tapered w/ Mitered Face	1	12.0	17.8
	2	14.0	21.8
	3	13.0	19.9

Design Performance Curves - 1-7'x6' RCB

Conclusion - Example Problem No. 1

Since the requirements called for the smallest possible reinforced concrete box culvert, the barrel should be a single 7' x 6'.

Selection of the inlet would be based on cost. The additional 1.3 ft. of FALL gains 62 cfs at AHW El. = 200.0, but this is not significant at this site. It appears that a side- or slope-tapered design meeting the Q and HW requirements of point 1 would be adequate and the least expensive.

Examination of the outlet control curve shows that a discharge of 1,200 cfs (20% above design) results in an AHW El. 5.5 ft. above design. At this site, no serious flooding of upstream property or the roadway will be caused by such a headwater, and no larger barrel is required.

PIPE CULVERT EXAMPLE NO. 2(a)

Given: Design Discharge (Q_{50}) = 150 cfs

Allowable Headwater Elevation = 100.0 ft.

Elevation Outlet Invert = 75.0 ft.

Culvert Length (L_a) = 350 ft.

Downstream channel approximates a 5 ft. wide trapezoidal channel with 2:1 side slopes and a Manning n of 0.03. S_o = 0.05

Requirements: This pipe culvert is located in suburban area where the AHW El. may be exceeded by 2 to 3 ft. without extreme damage. However, headwater elevations greater than 103.0 ft. should be avoided for flows significantly higher than the design Q of 150 cfs.

PROJECT: Example #2(a) | OUTLET CONTROL DESIGN CALCULATIONS | DESIGNER: JMN

STATION: ______ | DATE: 8-28-72

INITIAL DATA:

Q_{50} = 150 cfs

AHW El. = 100 ft.

S_o = 0.05

L_a = 350 ft.

El. Outlet Invert 75.0 ft.

Stream Data:

n = 0.030

S_o = 0.05

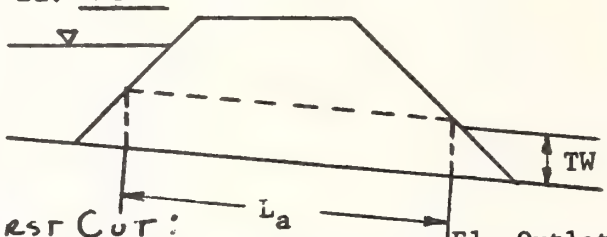
1

2

5'

Barrel Shape and Material Circular C.M.P.

SKETCH



First Cut: $Q = 150$, $L_a = 350'$, $k_e = 0.25$

$h_o \approx 5'$

$H \approx 100' - 75' - 5' = 20'$, ∴ D = 46", Try D = 42"

Q	Q/N	* H	Q/NB	(1) d_c	$\frac{d_c+D}{2}$	Qn	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. 1, N = 1, B = —, D = 3.5, k_e = 0.25											
150	150	31.	150	>3.5	3.5	—	1.6	3.5	109.5		75 + 31 + 3.5 = 109.5
											HW_o > AHW El., Try 48"
Trial No. 2, N = 1, B = —, D = 4', k_e = 0.25											
150	150	15.6	150	3.6	3.8	—	1.6	3.8	94.4		O.K.
100	100	7.6	100	3.1	3.5	—	1.4	3.5	85.5		Check Square Edge
200	200	27.8	200	>4	4.0	—	1.9	4.0	106.8		
Trial No. 3, N = 1, B = —, D = 4', k_e = 0.5											
150	150	16.2						3.8	95.0		From Inlet Control Section
100	100	7.2	Same as Trial #2					3.5	85.7		Calculations, Fall Reqd.
200	200	28.8						4.0	107.8		∴ Use Improved Inlet

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c + D}{2}$ or TW, whichever is larger.

(4) HW_o = H + h_o + El. Outlet Invert

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D.

SELECTED DESIGN

N = 1

B = — ft.

D = 4 ft.

k_e = 0.25 or 0.5

At Design Q:

HW_o = 94.4 ft.

V_o = ______ f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

PROJECT: Example # 2(a) — CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS — DESIGNER: J.M.N

STATION: ______ — DATE: 8-28-72

INITIAL DATA:

Q_{50} = 150 cfs

AHW El. = 100 ft.

S_o = 0.05

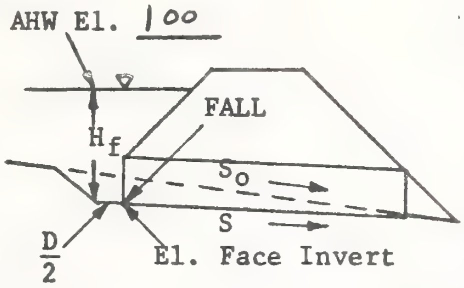
L_a = 350 ft.

El. Stream Bed at Face 92.5 ft.

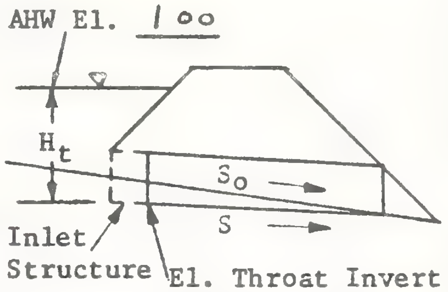
Barrel Shape and Material Circ. CMP

N = 1, B = —

D = 4', $NBD^{3/2}$ = 32.0 $= D^{5/2}$



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{N3}$ / ~~$\frac{Q}{NBD^{3/2}}$~~	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat / COMMENTS
Trial No. 1				**Inlet and Edge Description**						**Square Edges**
150	150	2.07	8.3	91.7	92.5	0.8	100.0	0.048		Fall Required, Use, Bevels
Trial No. 2				**Inlet and Edge Description**						**Beveled Edges**
150	150	1.92	7.7	92.3	92.5	~0	100.	0.05	16	Check Tapered Inlet
100	100	1.25	5.0	X	X	X	97.3	X		Throat
200	200	2.90	11.6	X	X	X	103.9	X		
Trial No. 3				**Inlet and Edge Description**						**Tapered Inlet Throat, Rough**
150	4.7	1.65	6.6	92.5 ~~93.4~~	92.5	0	99.1	0.05	16	Increases Q at
100	3.1	1.21	4.8	X	X	X	97.3	X		AHW El. = 100. to 170 cfs
200	6.2	2.22	8.9	X	X	X	101.4	X		

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL=El. Stream Bed at Face−El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed

(4) $S \approx S_o$ − FALL/L_a

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description: Bev Edges

FALL = 0 ft.

Invert El. = 92.5 ft.

Bevels:

Angle = 45°

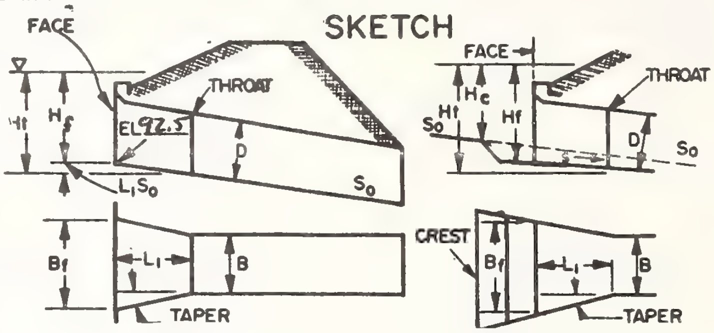
b = —— in., d = 2 in.

PROJECT: Example #2(a)	SIDE-TAPERED INLET DESIGN CALCULATIONS	DESIGNER: JMN
STATION: ______		DATE: 8-28-72

INITIAL DATA

Q_{50} = 150 cfs S_o = 0.05
AHW El. = 100 ft. L_a = 350 ft.
TAPER = 4 :1
Barrel Shape and Material: Circular CMP
Face Edge Description: 45° Bevels

SKETCH



N = 1, B = —, D = 4′

Q	El. Throat Invert	(1) ~~H_f/D~~ H_f/E	~~$Q/B_fD^{3/2}$~~ $Q/A_fE^{1/2}$	~~$D^{3/2}$~~ $E^{1/2}$	(2) ~~Min. B_f~~ Min. A_f	B_f	(3) L_1	(4) S	L_1S	(5) El. Face Invert	COMMENTS
Trial No. 1, Q = 150, HW_f = 99.1 (use lower column headings)											
150	92.5	1.4	4.0	2.0	18.8	6.0	4.0	0.05	0.2	92.7	~~$B_fD^{3/2}$~~ [or $A_fE^{1/2}$] = 18.85
											Std. Design: B_f=1.5 D
											=6′ ∴ Std. Design OK.
Trial No. ____, Q = ____, HW_f = ____											
											$B_fD^{3/2}$[or $A_fE^{1/2}$] = ____
Trial No. ____, Q = ____, HW_f = ____											
											$B_fD^{3/2}$[or $A_fE^{1/2}$] = ____

Notes and Equations: $(99.1 - 92.5 - 1)/4 = 1.4$

(1) H_f/D[or H_f/E] ≈ (HW_f-El. Throat Invert-1)/D [or E]
$D \le E \le 1.1D$

(2) Min. $B_f = Q\Big/\left[(D^{3/2})\dfrac{Q}{B_fD^{3/2}}\right]$

Min. $A_f = Q\Big/\left[(E^{1/2})\dfrac{Q}{A_fE^{1/2}}\right]$

(3) $L_1 = \left[\dfrac{B_f - NB}{2}\right]$ TAPER $= \left[\dfrac{6.0-4.0}{2}\right]4 = 4.0'$

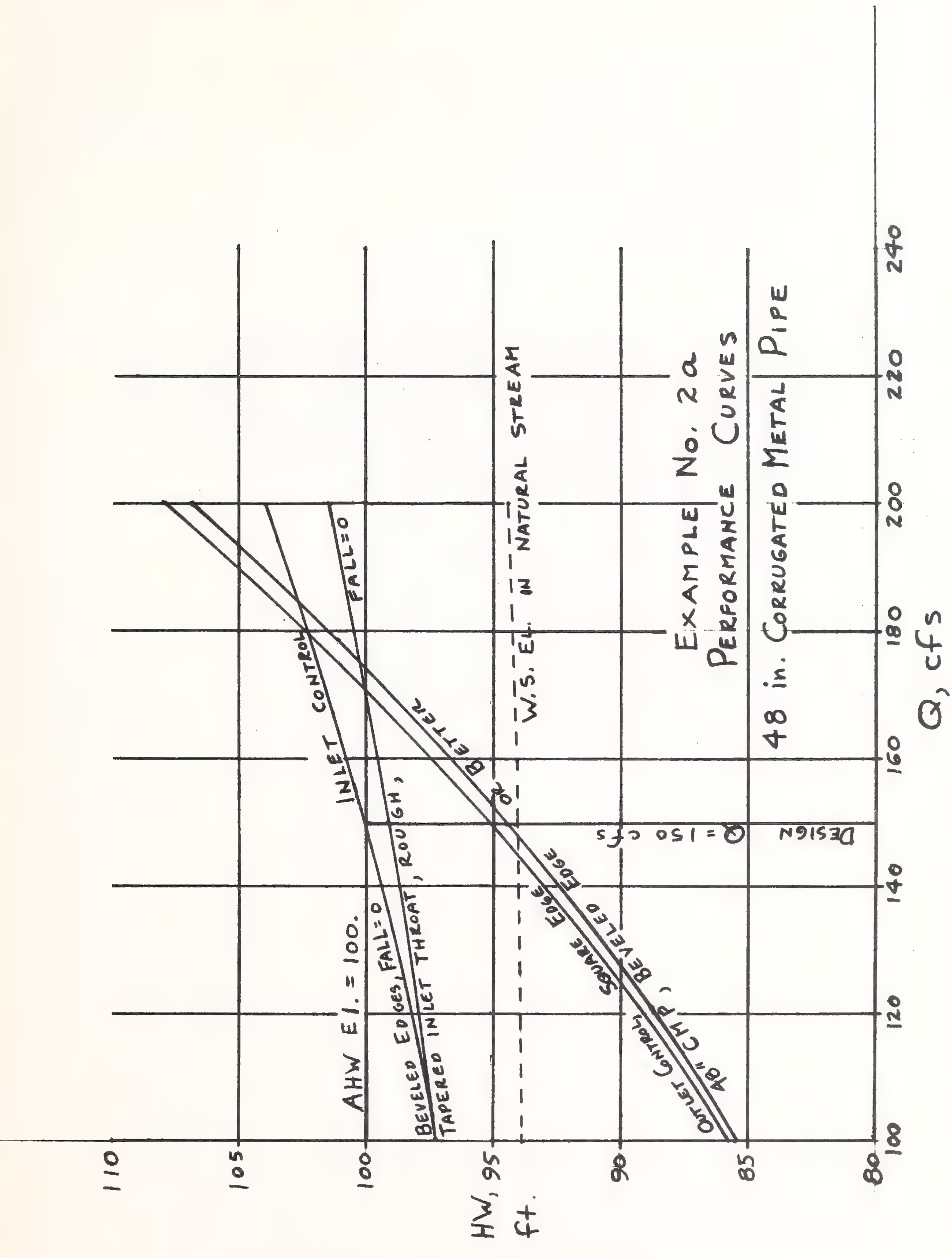
(4) From throat design

(5) El. Face Invert-El.Throat Invert>1 ft., recompute. Face and Throat may be lowered to better fit site.

SELECTED DESIGN

B_f = 6.0 ft.
L_1 = 4.0 ft.
Bevels: Angle 45 °
d = — in., b = 3 in.
Crest Check:
HW_c = 99.1 ft.
H_c = 6.1 ft.
Q/W = 44.0 (Chart 17)
Min.W = 3.4 ft.

$$\begin{array}{r} 99.1 \\ -93.0 \\ \hline 6.1 \end{array}$$


EXAMPLE No. 2a
PERFORMANCE CURVES
48 in. CORRUGATED METAL PIPE
Q, cfs
HW, ft.
AHW El. = 100.
BEVELED EDGES, FALL=0
TAPERED INLET THROAT, ROUGH,
INLET CONTROL
FALL=0
W.S. EL. IN NATURAL STREAM
OUTLET CONTROL, 48" CMP,
SQUARE EDGE
BEVELED EDGE
OR BETTER
DESIGN Q = 150 cfs
110
105
100
95
90
85
80
100
120
140
160
180
200
220
240

Conclusions: From the performance curves, beveled edges meet the AHW El. of 100 ft. and Q = 150 cfs, while the use of a side-tapered inlet would increase Q to 170 cfs at AHW El. = 100 ft. In both cases, the FALL =0. It appears that the beveled edge inlet would be sufficient and the least costly in this case, since the culvert performance curve does not exceed 103.0 ft. until Q is 186 cfs.

PIPE CULVERT EXAMPLE NO. 2(b)

Given: Same data as in Example No. 2, except AHW Elevation = 96.0 ft.

Requirements: Hydrological estimates are accurate and exceeding the AHW El. at higher discharges is not improtant at this site. Therefore, use the smallest barrel possible.

The outlet control curves of Problem 2(a) are applicable in this situation. The 48" C.M.P. is the smallest barrel which will meet AHW El. = 96.0 and Q = 150 cfs.

From the inlet control curves, it is clear that a FALL must be used on the tapered inlet to meet the AHW El. Try a side-tapered inlet, with FALL, and a slope-tapered inlet.

PROJECT: EXAMPLE #2(b) CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS

DESIGNER: J.M.N.

STATION: ______

DATE: 8-28-72

INITIAL DATA:

Q 50 = 150 cfs

AHW El. = 96.0 ft.

S_o = 0.05

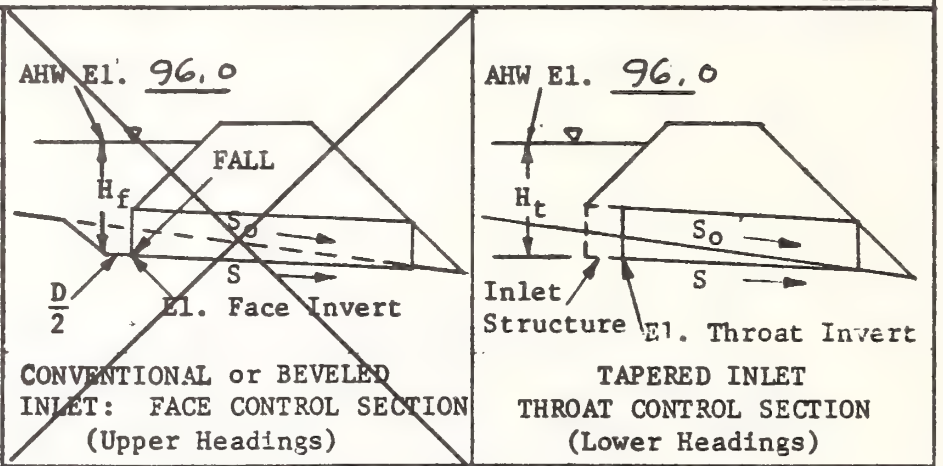
L_a = 350 ft.

El. Stream Bed at Face 92.5 ft.

Barrel Shape and Material CIRC. C.M.P.

N = 1, B = —

D = 4, $NBD^{3/2}$ = 32.0 = $D^{5/2}$



DEFINITIONS OF INLET CONTROL SECTION

Q	~~Q/NB~~ / $Q/NBD^{3/2}$	~~H_f/D~~ / H_t/D	~~H_f~~ / H_t	(1) ~~El. Face Invert~~ / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) ~~HW_f~~ / HW_t	(4) S	(5) V_o	Note: Use Upper Headings for Conventional or Beveled Face; Lower Headings for Tapered Inlet Throat. COMMENTS
Trial No. 1					Inlet and Edge Description					SMOOTH, TAPERED INLET THROAT, FALL = 2.8'
150	4.7	1.57	6.3	89.7	92.5	2.8	96.0	0.042	14	DECIDED THAT AT THIS
100	3.1	1.13	4.5				94.2			SITE, NO ADDITIONAL FALL
200	6.2	2.12	8.5				98.2			IS JUSTIFIED, DESIGN INLET
Trial No. 2					Inlet and Edge Description					FOR HW_f = 96.0, Q = 150 cfs.
150	4.7	1.65	6.6	89.4	92.5	3.1	96.0	0.041	14	ROUGH TAPERED INLET THROAT FALL = 3.1'
100	3.1	1.21	4.8				94.2			USE SMOOTH INLET FOR
200	6.2	2.22	8.9				98.3			SLOPE TAPERED, ROUGH FOR
Trial No. ____					Inlet and Edge Description					SIDE TAPERED

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL=El. Stream Bed at Face−El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed

(4) $S \approx S_o$ − FALL/L_a

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = 2.8 ft. or 3.1'

Invert El. = 89.7 ft. 89.4

Bevels: N/A

Angle = ____

b = ____ in., d = ____ in.

PROJECT: Example #2 (b)

STATION: ______

SIDE-TAPERED INLET
DESIGN CALCULATIONS

DESIGNER: J.M.N.

DATE: 8-28-72

INITIAL DATA

Q_{50} = 150 cfs S_o = 0.05

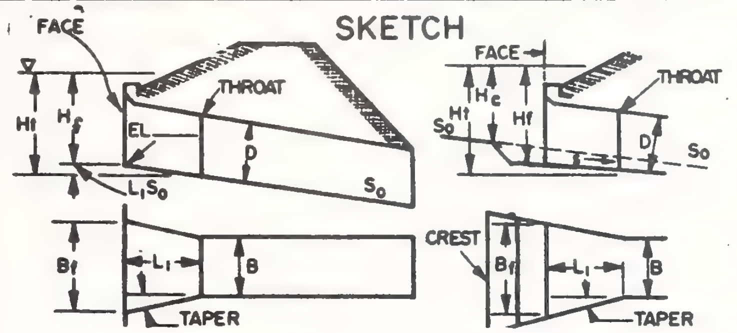
AHW El. = 96.0 ft. L_a = 350 ft.

TAPER = 4 :1

Barrel Shape and Material: Circular C.M.P.

Face Edge Description: 45° Bevels

SKETCH



N = 1, B = —, D = 4'

Q	El. Throat Invert	(1) ~~H_f/D~~ H_f/E	~~$Q/B_fD^{3/2}$~~ $Q/A_fE^{1/2}$	~~$D^{3/2}$~~ $E^{1/2}$	(2) ~~Min. B_f~~ Min. A_f	B_f	(3) L_1	(4) S	L_1S	(5) El. Face Invert	COMMENTS
Trial No. 1, Q = 150, HW_f = 96.0 (use lower column headings											
150	89.4	1.4	4.0	2.0	18.8	6.0	4.0	0.041	0.2	89.6	~~$B_fD^{3/2}$~~ [or $A_fE^{1/2}$] = 18.85
											CMP (rough) side
											tapered inlet
Trial No. ___, Q = ___, HW_f = ___											
											$B_fD^{3/2}$ [or $A_fE^{1/2}$] = ___
Trial No. ___, Q = ___, HW_f = ___											
											$B_fD^{3/2}$ [or $A_fE^{1/2}$] = ___

Notes and Equations: (96.0 − 89.4 − 1)/4 = 1.4

(1) H_f/D [or H_f/E] = (HW_f-El. Throat Invert-1)/D [or E]
$D \le E \le 1.1D$

(2) Min. $B_f = Q/\left[(D^{3/2})\ \frac{Q}{B_fD^{3/2}}\right]$

Min. $A_f = Q/\left[(E^{1/2})\ \frac{Q}{A_fE^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER $= \left[\frac{6.0-4.0}{2}\right] 4 = 4.0$

(4) From throat design

(5) El. Face Invert-El.Throat Invert>1 ft., recompute. Face and Throat may be lowered to better fit site.

SELECTED DESIGN

B_f = 6.0 ft.

L_1 = 4.0 ft.

Bevels: Angle ___ °

d = — in., b = 3 in.

Crest Check:

HW_c = 96.0 ft.

H_c = 3.0 ft.

Q/W = 15. (Chart 17)

Min.W = 10.0 ft.

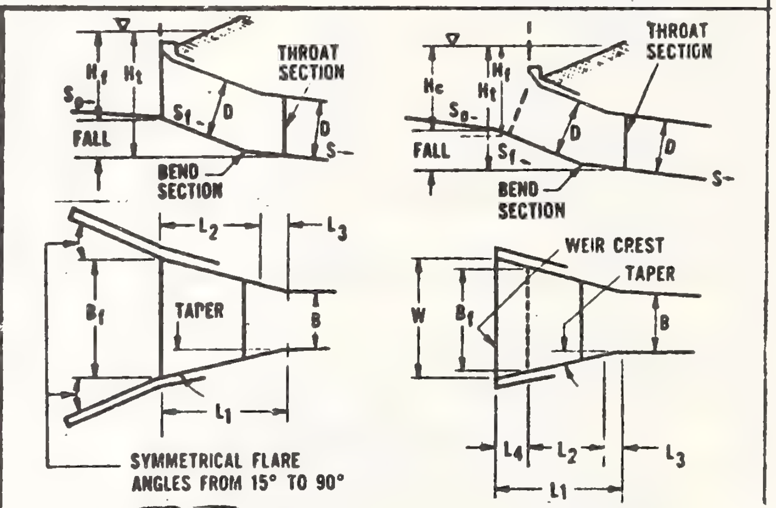
$$\begin{array}{r} 96.0 \\ -93.0 \\ \hline 3.0 \end{array}$$

PROJECT: EXAMPLE #2(b) STATION: ________	SLOPE-TAPERED INLET DESIGN CALCULATIONS	DESIGNER: J.M.N. DATE: 8-28-72

INITIAL DATA

Q= 150 cfs S_o = 0.05
AHW El. = 96.0 ft. L_a = 350 ft.
El. Stream bed at crest ____ ft.
El. stream bed at face 92.5 ft.
TAPER = 4 :1 (4:1 to 6:1)
S_f = 2 :1 (2:1 to 3:1)
Barrel Shape and Material CIRCULAR C.M.P.
Inlet Edge Description BEVELED

NOTE: USE SQUARE TO CIRCULAR TRANSITION SECTION, D = B = 4'



VERTICAL   MITERED

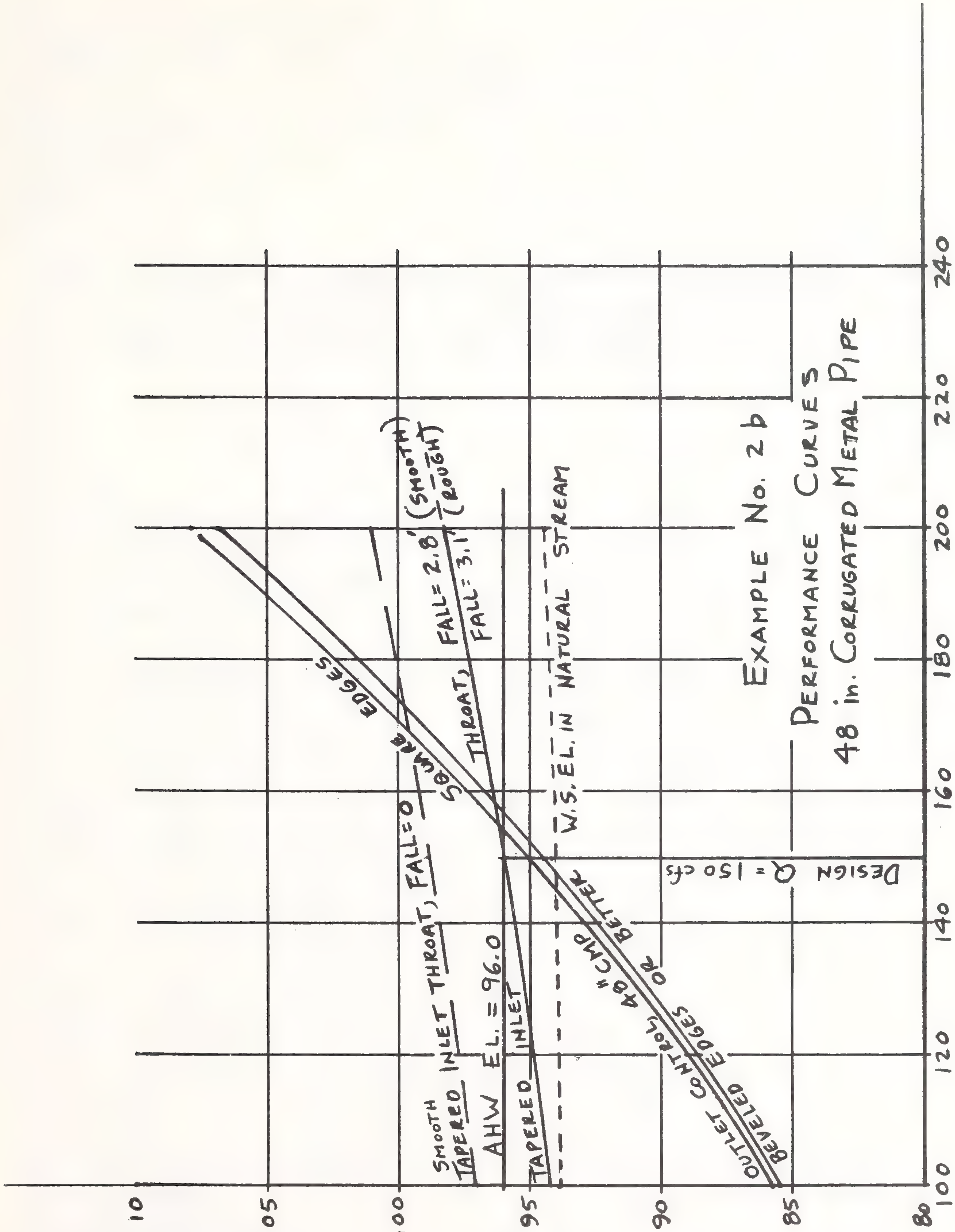
N = 1 , B = — , D = 4' (SMOOTH CONCRETE INLET)

	Q	HW_f	El. Throat Invert	(1) El. Face Invert	(2) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3) Min. B_f	B_f	S	COMMENTS
Trial 1	150	96.0	89.7	92.5	3.5	0.88	2.4	8.0	7.8	8.0	0.042	$B_f D^{3/2}$ = ____
												VERTICAL FACE,
												MIN. FALL REQUIRED
Trial 2												$B_f D^{3/2}$ = ____

Note: Use only throat designs with FALL > 0.25D
(1) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − 0.4D (Approx.), but higher than throat invert elevation.
(2) $H_f = HW_f$ − El. face invert
(3) Min $B_f = Q / [(D^{3/2})\ Q/B_f D^{3/2}]$

(4) Min. L_3	(5) L_4	(6) L_2	(7) Check L_2	(8) Adj. L_3	(9) Adj. TAPER	(10) L_1	(11) W	$\frac{Q}{W}$	H_c	(12) Max. Crest El.	GEOMETRY
2.0	—	5.6 <	6.0	2.4	—	8.0	—	—	—	—	B_f = 8.0 ft. L_3 = 2.8 ft. L_1 = 8.0 ft. L_4 = — ft. L_2 = 5.6 ft. TAPER = 4 :1

(4) Min $L_3 = 0.5B$ = 0.5(4) = 2.0
(5) $L_4 = S_{fy} + D/S_f$ N/A
(6) L_2 = (El. Crest Invert − El. Throat Invert)$S_f - L_4$ = (92.5 − 89.7) 2 = 5.6'
(7) Check $L_2 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_3
(8) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER − L_2
(9) If (6)>(7) Adj. TAPER = $(L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$
(10) $L_1 = L_2 + L_3 + L_4$
(11) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$
(12) Max. Crest El = $HW_f - H_c$



EXAMPLE No. 2b
PERFORMANCE CURVES
48 in. CORRUGATED METAL PIPE
Q, cfs
HW, ft.
100
120
140
160
180
200
220
240
80
85
90
95
100
105
110
SQUARE EDGES
BEVELED EDGES
OUTLET CONTROL, 48" CMP OR BETTER
SMOOTH TAPERED INLET THROAT, FALL = 0
TAPERED INLET THROAT, FALL = 2.8' (SMOOTH)
FALL = 3.1' (ROUGH)
AHW EL. = 96.0
W.S. EL. IN NATURAL STREAM
DESIGN Q = 150 cfs

Conclusions: Selection of side-tapered or slope-tapered inlet must be based on economics, as either will perform the required function. Additional FALL is not warranted at this site. Face design was selected to pass 150 cfs at AHW El. = 96.0.

PROJECT: __________

STATION: __________

OUTLET CONTROL DESIGN CALCULATIONS

DESIGNER: __________

DATE: __________

INITIAL DATA:

Q______ = ________ cfs

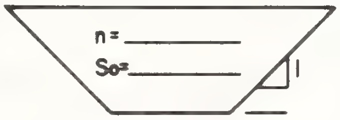
AHW El. = ________ ft

S_o = ________

L_a = ________ ft.

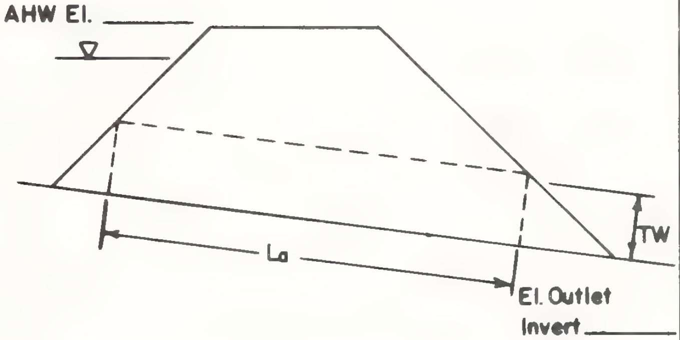
El. Outlet Invert ______ ft.

Stream Data:



Barrel Shape and Material ______________________

SKETCH



Q	$\frac{Q}{N}$	* H	$\frac{Q}{NB}$	(1) d_c	$\frac{d_c+D}{2}$	Q_n	(2) TW	(3) h_o	(4) HW_o	(5) V_o	COMMENTS
Trial No. ____, **N = ____, B = ____, D = ____, k_e = ____											
Trial No. ____, **N = ____, B = ____, D = ____, k_e = ____											
Trial No. ____, **N = ____, B = ____, D = ____, k_e = ____											

Notes and Equations:

(1) d_c cannot exceed D

(2) TW based on d_n in natural channel, or other downstream control.

(3) $h_o = \frac{d_c+D}{2}$ or TW, whichever is larger.

(4) $HW_o = H + h_o +$ El. Outlet Invert.

(5) Outlet Velocity (V_o) = Q/Area defined by d_c or TW, not greater than D. Do not compute until control section is known.

SELECTED DESIGN

N = ________ At Design Q:

B = ________ ft.

D = ________ ft. HW_o = ________ ft.

k_e = ________ V_o = ________ f/s

$$* \quad H = \left[1 + k_e + \frac{29n^2 \cdot L}{R^{1.33}}\right] \frac{V^2}{2g}$$

** Number of Barrels

PROJECT: ______	CULVERT INLET CONTROL SECTION DESIGN CALCULATIONS	DESIGNER: ______
STATION: ______		DATE: ______

INITIAL DATA:

Q ______ = ______ cfs

AHW El. = ______ ft.

S_o = ______

L_a = ______ ft.

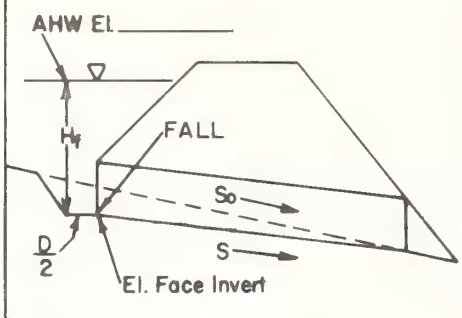
El. Stream Bed at Face ______ ft.

Barrel Shape and Material ______

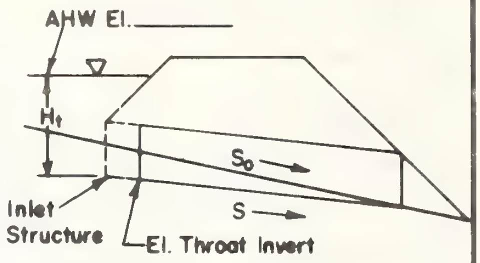
N = ______, B = ______

D = ______, $NBD^{3/2}$ = ______

(Pipe) $ND^{5/2}$ = ______



CONVENTIONAL or BEVELED INLET: FACE CONTROL SECTION (Upper Headings)



TAPERED INLET THROAT CONTROL SECTION (Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	$\frac{Q}{NB}$ / $\frac{Q}{NBD^{3/2}}$	$\frac{H_f}{D}$ / $\frac{H_t}{D}$	H_f / H_t	(1) El. Face Invert / El. Throat Invert	El. Stream Bed At Face	(2) FALL	(3) HW_f / HW_t	(4) S	(5) V_o	Note: Use Upper Headings for Coventional or Beveled Face; Lower Headings for Tapered Inlet Throat. / COMMENTS
Trial No. ______ Inlet and Edge Description ______										
Trial No. ______ Inlet and Edge Description ______										
Trial No. ______ Inlet and Edge Description ______										

Notes and Equations:

(1) El. Face (or throat) invert = AHW El. − H_f (or H_t)

(2) FALL = El. Stream Bed at Face − El. face (or throat) invert

(3) HW_f (or HW_t) = H_f (or H_t) + El. face (or throat) invert, where El. face (or throat) invert should not exceed El. stream bed.

(4) $S \approx S_o - FALL/L_a$

(5) Outlet Velocity = Q/Area defined by d_n at S

SELECTED DESIGN

Inlet Description:

FALL = ______ ft.

Invert El. = ______ ft.

Bevels:

Angle = ______

b = ______ in., d = ______ in.

PROJECT: ______________ DESIGNER: ______________

SIDE-TAPERED INLET
DESIGN CALCULATIONS

STATION: ______________ DATE: ______________

INITIAL DATA

Q______ = ______ cfs S_o = ______

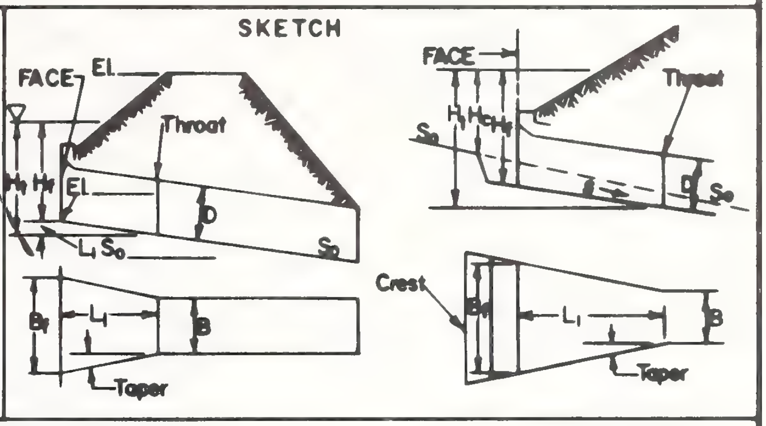
AHW El. = ______ ft. L_a = ______ ft.

TAPER = ______ : 1

Barrel Shape and Material ______________

Face Edge Description ______________

SKETCH



N= ______ , B= ______ , D= ______

Q	El. Throat Invert	(1) H_f/D / H_f/E	$\frac{Q}{B_f D^{3/2}}$ / $\frac{Q}{A_f E^{1/2}}$	$D^{3/2}$ / $E^{1/2}$	(2) Min. B_f / Min. A_f	B_f	(3) L_1	(4) S	$L_1 S$	(5) El. Face Invert	COMMENTS
Trial No. ______ , Q = ______ , HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______
Trial No. ______ , Q = ______ , HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______
Trial No. ______ , Q = ______ , HW_f = ______											
											$B_f D^{3/2}$ [or $A_f E^{1/2}$] = ______

Notes and Equations:

(1) H_f/D [or H_f/E] = (HW_f − El. Throat Invert − 1) / D [or E]

$D \leq E \leq 1.1D$

(2) Min. $B_f = Q / (D^{3/2}) \left[\frac{Q}{B_f D^{3/2}}\right]$

Min. $A_f = Q / (E^{1/2}) \left[\frac{Q}{A_f E^{1/2}}\right]$

(3) $L_1 = \left[\frac{B_f - NB}{2}\right]$ TAPER

(4) From throat design

(5) El. Face Invert − El. Throat Invert > 1 ft., recompute. Face and Throat may be lowered to better fit site.

SELECTED DESIGN

B_f = ______ ft.

L_1 = ______ ft.

Bevels: Angle ______ °

d = ______ in., b = ______ in.

Crest Check:

HW_c = ______ ft.

H_c = ______ ft.

Q/W = ______ (Chart 17)

Min. W = ______ ft.

PROJECT: ____________ SLOPE-TAPERED INLET DESIGN CALCULATIONS DESIGNER: ____________

STATION: ____________ DATE: ____________

INITIAL DATA:

Q_____ = cfs = S_o = ________

AHW EL. _____ ft. La = ________ ft.

El. Stream bed at crest ________ ft.

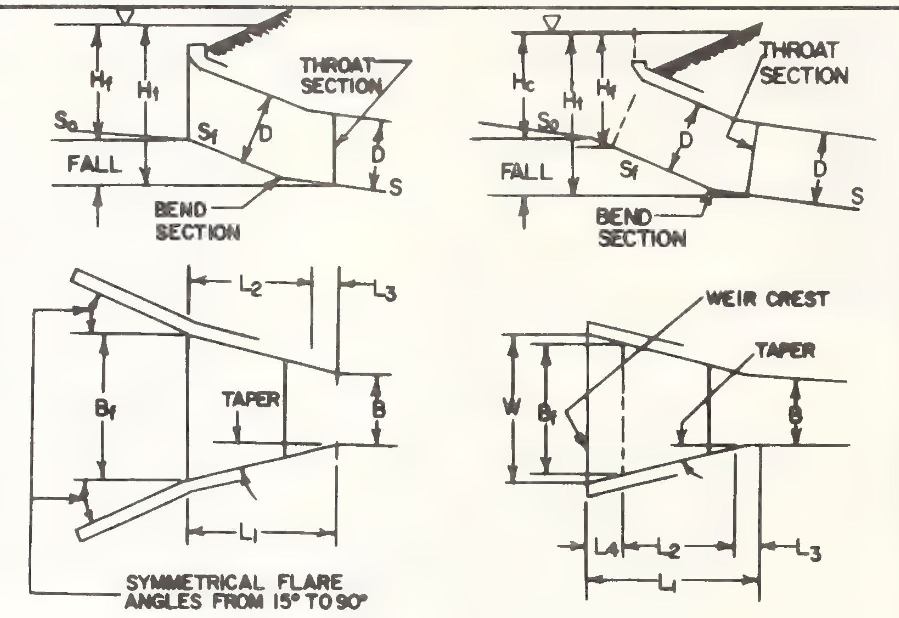
El. stream bed at face ________ ft.

TAPER = ___:1 (4:1 to 6:1)

S_f = _____:1 (2:1 to 3:1)

Barrel Shape and Material ____________

Inlet Edge Description ____________



VERTICAL MITERED

N= _____ , B= _____ , D= _____

	Q	HW_f	El. Throat Invert	(1.) El. Face Invert	(2.) H_f	$\frac{H_f}{D}$	$\frac{Q}{B_f D^{3/2}}$	$D^{3/2}$	(3.) Min. B_f	B_f	S	Comments
Trial 1												$B_f D^{3/2}$ = ________
Trial 2												$B_f D^{3/2}$ = ________

Note: Use only throat designs with FALL > 0.25D

(1.) El. face invert: Vertical = Approx. stream bed elevation
Mitered = El. Crest − 0.4D (Approx.), but higher than throat invert elevation.

(2.) $H_f = HW_f$ − El. face invert

(3.) Min. $B_f = Q/(D^{3/2})$ $Q/(B_f D^{3/2})$

(4.) Min. L_3	(5.) L_4	(6.) L_2	(7.) Check L_2	(8.) Adj L_3	(9.) Adj TAPER	(10.) L_1	(11.) W	$\frac{Q}{W}$	H_c	(12.) Max. Crest El.	GEOMETRY
											B_f = ___ft. L_3= ___ft. L_1 = ___ft. L_4= ___ft. L_2= ___ft. d = ___in. b= ___in. TAPER= _____:1

(4.) Min. L_3 = 0.5B

(5.) $L_4 = S_f y + D/S_f$

(6.) L_2 = (El. Crest Invert − El. Throat Invert) $S_f - L_4$

(7.) Check $L = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_3$

(8.) If (7)>(6), Adj. $L_3 = \left[\frac{B_f - NB}{2}\right]$ TAPER $- L_2$

(9.) If (6)>(7) Adj. TAPER $= (L_2 + L_3) / \left[\frac{B_f - NB}{2}\right]$

(10.) $L_1 = L_2 + L_3 + L_4$

(11.) Mitered: $W = NB + 2\left[\frac{L_1}{TAPER}\right]$

(12.) Max. Crest El. = $HW_f - H_c$

4.12 IRRIGATION SIPHONS

General

Low roadway grades at an irrigation crossing will often times make it impossible to provide the required cover over the irrigation pipe. When this happens it is necessary to use an inverted siphon. A conventional culvert crossing should always be designed and the cover checked to be sure a siphon is required. Because of the additional cost and maintenance associated with siphons, they should only be used when necessary.

The siphon detail shown in Figure 4.24 should be followed as closely as possible. Conditions will arise which will require deviations from this standard but they should be kept to a mininum. The inlet and outlet structures and the trash racks (if required) are detailed on Form HYD-3 and the "Trash-guard for Concrete Irrigation Inlet and Outlet Transition Structures" as shown in the Book of Standard Drawings.

The barrel of the siphon can be constructed out of either round corrugated steel pipe or round reinforced concrete pipe. The concrete pipe, because of its much more favorable roughness coefficient, is normally used. A corrugated steel barrel should be used only when the soil is reactive to concrete, when the ditch company requests it, or when the amount of head loss is not critical.

All of the joints in the siphon should be water tight and should conform to the "Standard Specifications for Road and Bridge Construction". If the head of the siphon exceeds fifteen (15) feet, a special joint design is necessary.

Trash racks should be considered on all siphon installations. There are two possible reasons why trash racks might be required: (1) Because of their shape, siphons are extremely susceptible to plugging by floating debris. If any floating debris is expected, a trash rack should be placed on the inlet

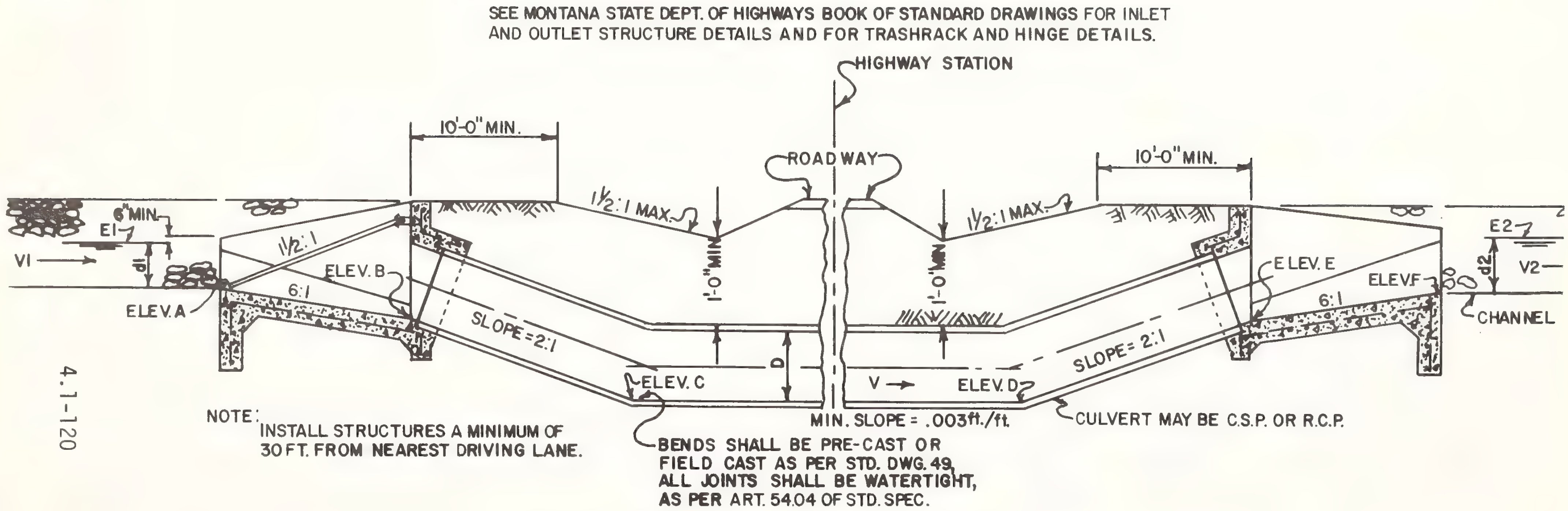

SEE MONTANA STATE DEPT. OF HIGHWAYS BOOK OF STANDARD DRAWINGS FOR INLET AND OUTLET STRUCTURE DETAILS AND FOR TRASHRACK AND HINGE DETAILS.
HIGHWAY STATION
10'-0" MIN.
ROADWAY
1½:1 MAX.
10'-0" MIN.
1½:1 MAX.
6" MIN.
E1
V1
d1
1½:1
ELEV. B
6:1
ELEV. A
1'-0" MIN
1'-0" MIN
SLOPE = 2:1
D
ELEV. C
V
ELEV. D
MIN. SLOPE = .003 ft./ft.
SLOPE = 2:1
E LEV. E
ELEV. F
6:1
E2
d2
V2
CHANNEL
CULVERT MAY BE C.S.P. OR R.C.P.
NOTE: INSTALL STRUCTURES A MINIMUM OF 30 FT. FROM NEAREST DRIVING LANE.
BENDS SHALL BE PRE-CAST OR FIELD CAST AS PER STD. DWG. 49. ALL JOINTS SHALL BE WATERTIGHT, AS PER ART. 54.04 OF STD. SPEC.

FIGURE 4.24

structure. (2) Siphons are quite dangerous to children and livestock. If there is a chance that children or livestock will be near the canal or siphon, then trash racks should be placed on both the inlet and outlet structure for safety purposes. If the inlet and outlet structure shown on Form HYD-3 are used, the trash rack shown in the Book of Standard Drawings may be used.

Design Procedure

The design procedure presented here should be used in conjunction with Figure 4.24 and Form HYD-3. The design procedure hinges on the computation of the head losses for several barrel diameters and the selection of the barrel diameter which will hydraulically meet the upstream and downstream canal control water surface elevations.

Step 1

Determine the normal water depth (d_1), velocity (V_1), and water surface elevation (E_1) at the inlet end of the siphon and the normal water depth (d_2), velocity (V_2), and water elevation (E_2) at the outlet end of the siphon.

Step 2

Compute the head losses for several barrel diameters in the range that will be used. The total head loss for each barrel should include the entrance loss, exit loss, friction loss, the losses through the two bends, and the trash rack losses if trash racks are used. See Section 4.16 for a discussion of an ITF computer program which will do this computation. The head losses are discussed in detail below:

Entrance Loss - The entrance is dependent upon the transition structure and the change in velocity between the canal and the siphon barrel.

$$h_i = K_e \frac{V^2 - V_1{}^2}{2g} \qquad \text{Loss in ft.}$$

where K_e = .3 for transition structure shown on Form HYD-3.

V = Velocity in siphon barrel (ft./sec.)

V_1 = Velocity in approach canal (ft./sec.)

$g = 32.2$ ft./sec.^{2}

Exit Loss - The exit loss is also dependent upon the transition structure and the difference in velocity heads between the siphon and the downstream canal.

$$h_o = K_e \frac{V^2 - V_2{}^2}{2g} \qquad \text{Loss in ft.}$$

where K_e = .4 for transition structure shown on Form HYD-3.

V = Velocity in siphon barrel (ft./sec.)

V_2 = Velocity in downstream canal (ft./sec.)

$g = 32.2$ ft./sec.^{2}

Barrel Friction Loss - The barrel friction losses are computed from a form of Mannings Equation.

$$h_f = 4.66\, n^2 \frac{L\, Q^2}{D^{16/3}} \qquad \text{Loss in ft.}$$

where N = roughness coefficient, n = .012 for concrete
n = .024 for corrugated steel

L = Total length of siphon barrel (ft.)

Q = Canal flow (ft^3/sec.)

D = Pipe diameter (ft.)

Bend Losses - The bend losses are dependent on the velocity in the siphon and the degree of bend. The total loss for the two bends is computed in the following formula:

$$h_b = 2\, K_b \frac{V^2}{2g} \qquad \text{Loss in ft.}$$

where K_b = bend coefficient = .12 for a 27 degree bend

V = Velocity in siphon barrel (ft./sec.)

$g = 32.2$ ft/sec.^{2}

Trash Rack Losses - If trash racks are used on either or both ends of the siphon, the loss through them must be computed. This loss is dependent on the trash rack geometry and the velocity through the rack. For design purposes, the velocity in the siphon barrel should be used as the velocity through the rack. The trash rack losses can be calculated by the following formula:

$$h_{tr} = 0.11 \left(\frac{TV^2}{d}\right) (\text{Sin } A) (\text{Sec}^{15/8} B) \quad \text{Loss in ft.}$$

where
T = Thickness of trash rack bars (in.)
V = Water velocity below trash rack (ft./sec.)
A = Angle of inclination of rack with horizontal
B = Angle of approach
d = Center to center spacing of trash rack bars (in.)

The solution of this equation is provided by the nomograph of Figure 4.25. "Trashguard for Concrete Irrigation Inlet and Outlet Transition Structures" as shown in the Book for Standard Drawings provides the geometry for the standard trash rack.

Total Head Loss -

$$h_t = h_i + h_o + h_f + h_b + (h_{tr} \text{ x No. of racks})$$

Step 3

Compute the available head in the existing canal. This is the difference between water surface elevations at the inlet and outlet ends of the siphon ($E_1 - E_2$).

Step 4

Choose the smallest barrel diameter for which the computed head losses (from Step 2) are equal to or less than the available head (from Step 3). This is the first trial barrel diameter.

Step 5

The first trial size determined in Step 4 will provide an effective crossing, but it is usually not the most economical size. It is possible, in most situations, to design a siphon that is hydraulically adequate and more economical than the first trial size. This design requires that the available head be increased. The following methods of increasing the available head should be investigated and included in the design if justified.

(1) The most common method used to increase the available head is to raise the upstream water surface elevation, (El.), with backwater. The backwater created by a siphon shall be limited to 0.3 feet. When backwater is used as a method of increasing the available head, the designer shall investigate the existing system to insure that there will still be adequate freeboard.

(2) The second method used to increase the available head is to lower the downstream water surface either by changing the channel section or channel slope by realignment. The designer must investigate all existing irrigation facilities within the limits of the change to insure that any canal structure (check, turnouts, etc.) will still function properly.

After establishing the water surface elevations, compute the new available head as explained in Step 3.

Step 6

Check to insure that the cost of any required channel excavation or of replacing any canal structures is justified by the savings for the siphon.

Step 7

Once the barrel diameter has been established, the final details for the plans shall be prepared. The siphon summary sheet, Form HYD-3, shall be completed showing all dimensions, elevations, and references necessary to

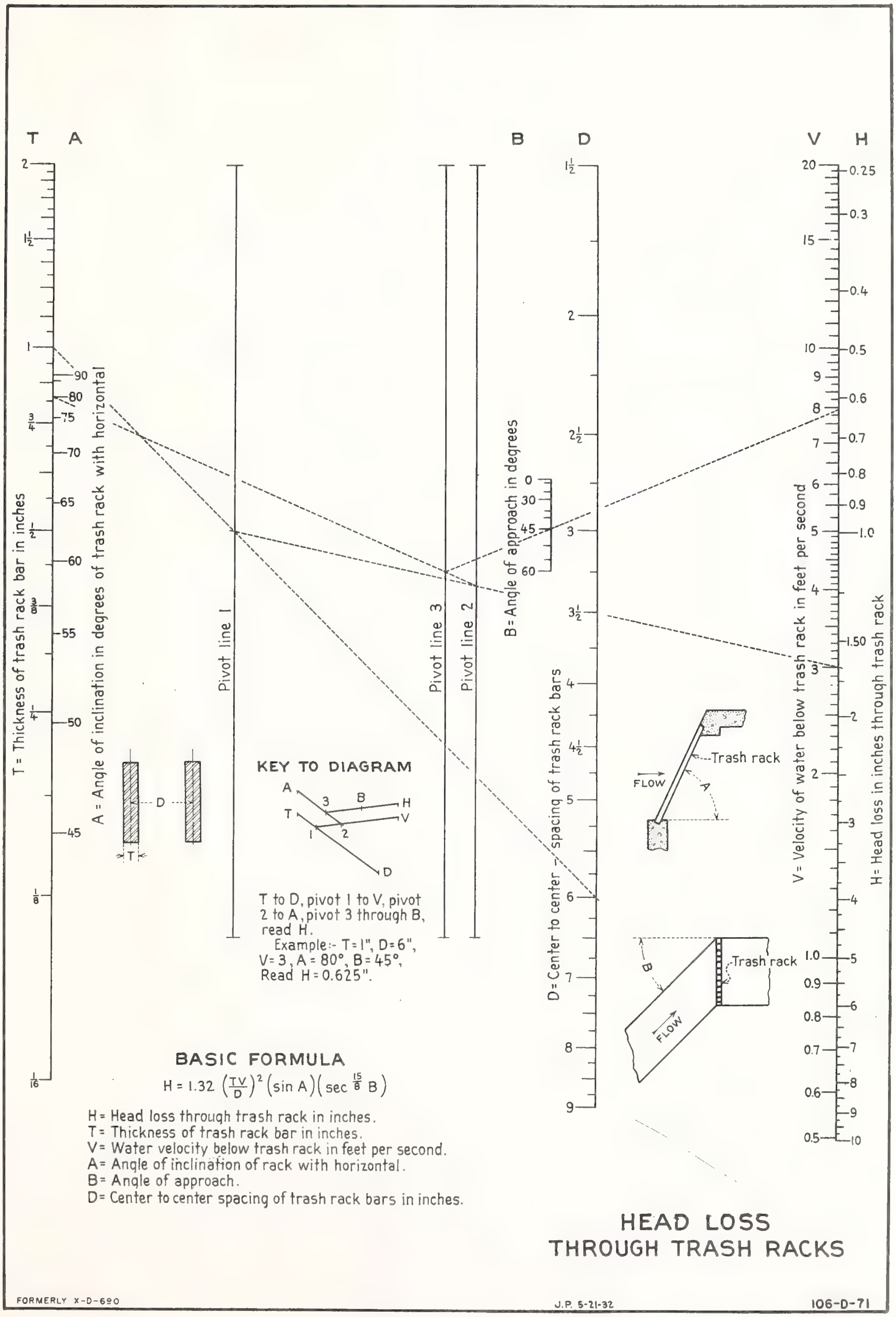

T
A
B
D
V
H
T = Thickness of trash rack bar in inches
A = Angle of inclination in degrees of trash rack with horizontal
Pivot line 1
Pivot line 3
Pivot line 2
B = Angle of approach in degrees
D = Center to center spacing of trash rack bars
V = Velocity of water below trash rack in feet per second
H = Head loss in inches through trash rack
Trash rack
FLOW
KEY TO DIAGRAM
T to D, pivot 1 to V, pivot 2 to A, pivot 3 through B, read H.
Example:- T = 1", D = 6", V = 3, A = 80°, B = 45°, Read H = 0.625".
BASIC FORMULA
$H = 1.32 \left(\frac{TV}{D}\right)^2 (\sin A)\left(\sec^{\frac{15}{8}} B\right)$
H = Head loss through trash rack in inches.
T = Thickness of trash rack bar in inches.
V = Water velocity below trash rack in feet per second.
A = Angle of inclination of rack with horizontal.
B = Angle of approach.
D = Center to center spacing of trash rack bars in inches.
HEAD LOSS
THROUGH TRASH RACKS
FORMERLY X-D-690
J.P. 5-21-32
106-D-71

Fig. 4.25

describe the size and shape of the structure and its location. All dimensions shown on Form HYD-3 must be in accordance with the maximums and minimums given on Figure 4.24.

4.13 OPEN BOTTOM CORRUGATED STEEL ARCHES

4.14 STRUCTURAL REQUIREMENTS OF CULVERTS

Once a culvert has been sized according to its hydraulic capabilities it is necessary to determine its structural requirements. Structural requirements for corrugated steel pipe, open bottom corrugated steel arches and reinforced concrete pipe culverts will be handled separately in this section.

Corrugated Steel Pipe

Metal thickness is the principle measure of strength in corrugated steel pipe. The required metal thickness depends on the pipe size, shape of pipe, height of fill and dimensions of corrugations. The relationships between these factors are shown in Tables 4.2, 4.3, 4.4, 4.5, 4.6, and 4.7. The tables show the minimum and the maximum permissible fill heights for each combination of pipe size and metal thickness. Fill height is measured from the top of the pipe to the finished roadway surface.

When there is a choice between two sizes of corrugations, the one which is most economical for the particular situation should be used. However, the corrugation dimensions should not be specified. It is assumed that when the metal thickness and height of fill (cover) are specified, the contractor will use the most economical corrugation size that will fit the situation.

Most culvert installations should be "round" pipe. Pipe arches should be used only where headroom (cover over pipe) is limited or where local conditions make the flatter shape of the pipe arch more effective for carrying water.

Corrugated steel pipe develops its strength because of its ability to deflect vertically under load without distress so that the sides of the pipe move outward against the fill. This outward movement develops a resisting pressure in the fill normal to the sides of the pipe that tends to equal the pressures acting on the top and bottom so that all of the forces become more

evenly distributed and bending stresses are considerably relieved. However, if the pipe deflects too far, bending stresses again develop and failure can occur. It is possible to provide a 5% elongation of the vertical axis during manufacture. This elongation will allow greater deflection and, therefore, allows the fill to carry a greater part of the load. This gives the pipe greater strength and allows higher fill heights. The allowable fill heights for elongated pipe are shown in parentheses on Tables 4.2, 4.3, and 4.4. A 5% elongation should only be called for when it will allow the use of a smaller metal thickness.

There are two theories in use for determining the allowable fill heights for corrugated steel pipe. One is based on deflection of the pipe and the other is based on ring compression. The tables presented are computed by the deflection method as presented in "Corrugated Metal Pipe - Structural Design Criteria and Recommended Installation Practice", published by the Federal Highway Administration. The Tables include a factor of safety of at least 3.33.

Tables 4.2, 4.3, 4.4, 4.5, 4.6, and 4.7 are based on H20 highway loading. When it is necessary to place a culvert under a railroad, a request should be sent to the railroad asking for their structural requirements.

FILL HEIGHT TABLE 4.2

For Corrugated Steel Pipe, 2 2/3-inch by 1/2 inch Corrugations, Riveted, Welded, or Helical Fabrications, H20 Loading

Pipe Diameter (Inches)	Minimum Cover, Top of Pipe to Top of Sub-grade (inches)	Maximum Fill Heights Above Top of Pipe in Feet				
		Metal Thickness in Inches				
		0.064	0.079	0.109	0.138	0.168
12	12	84	91	----------	----------	----------
15	12	67	73	----------	----------	----------
18	12	56	61	----------	----------	----------
24	12	42	46	59	----------	----------
30	12	34	36	47	----------	----------
36	12	28	30	39	41	----------
42	12	31	43	46(67)	48(70)	50(73)
48	12	27	37	45(58)	46(61)	47(64)
54	12	----------	33	43(52)	44(54)	45(57)
60	12	----------	----------	43(47)	43(49)	44(51)
66	12	----------	----------	42	43	43(47)
72	12	----------	----------	----------	41	43
78	12	----------	----------	----------	----------	39
84	12	----------	----------	----------	----------	35

Values for elongated pipe are shown in parentheses

FILL HEIGHT TABLE 4.3

Corrugated Steel Pipe, 3-inch by 1-inch Corrugations, Riveted, Welded, Helical, or Bolted Fabrication, H20 Loading

Pipe Diameter (inches)	Minimum Cover, Top of Pipe to Top of Sub-grade (inches)	Maximum Fill Heights Above Top of Pipe in Feet				
		Metal Thickness in Inches				
		0.064	0.079	0.109	0.138	0.168
36	12	48	60	78(88)	89(106)	101(118)
42	12	41	51	64(76)	71(91)	79(101)
48	12	36	45	57(66)	61(80)	66(88)
54	12	32	40	52(59)	55(71)	59(79)
60	12	29	36	49(53)	51(64)	54(71)
66	12	26	33	47	49(58)	51(64)
72	12	24	30	44	47(53)	49(59)
78	12	22	28	41	46(49)	47(54)
84	12	21	26	38	45	46(51)
90	12	19	24	35	43	45
96	12	18	22	33	40	44
102	24	17	21	31	38	42
108	24	----------	20	30	35	39
114	24	----------	19	28	34	37
120	24	----------	----------	27	32	35

Values for elongated pipe are shown in parentheses

FILL HEIGHT TABLE 4.4

Structural Steel Plate Pipe, 6-inch by 2-inch Corrugations
Bolted Fabrication, H20 Loading

Pipe Diameter (inches)	Minimum Cover, Top of Pipe to Top of Sub-grade (inches)	Maximum Fill Heights Above Top of Pipe in Feet						
		Metal Thickness in Inches						
		0.109	0.138	0.168	0.188	0.128	0.249	0.280
60	12	43	62	81	93	106(111)	116(132)	126(144)
72	12	36	52	68	37(78)	79(93)	85(110)	91(120)
84	12	31	44	58	61(67)	65(79)	69(94)	72(103)
96	12	27	39	51	55(58)	57(69)	60(82)	62(90)
108	24	24	34	45	50	52(62)	54(73)	56(80)
120	24	22	31	41	47	49(56)	50(66)	52(72)
132	24	20	28	37	42	47(51)	48(60)	49(66)
144	24	18	26	34	39	45	46(55)	47(60)
156	24	17	24	31	36	43	45(50)	46(56)
168	24	15	22	29	33	40	44(47)	45(52)
180	24	14	21	27	31	37	44	44(48)
192	24	-------	19	25	29	35	41	43
204	36	-------	18	23	27	33	39	43
216	36	-------	--------	21	26	31	37	30
228	36	-------	--------	------	25	29	35	38
240	36	-------	--------	------	23	28	33	36
252	36	-------	-------	------	-------	27	31	34

Values of elongated pipe are shown in parentheses

FILL HEIGHT TABLE 4.5

Corrugated Steel Pipe Arches, 2 2/3-inch by 1/2-inch Corrugations,
Riveted, Welded, or Helical Fabrication, H-20 Loading

Pipe Dimensions Span x Rise (inches)	Minimum Cover, Top of Pipe to Top of Subgrade (inches)	Minimum Thickness Required (inches)	Maximum Fill Heights Above Top of Pipe (in ft.)
17 x 13	18	0.064	31
21 x 15	18	0.064	12
24 x 18	18	0.064	10
28 x 20	18	0.064	10
35 x 24	18	0.064	9
42 x 29	18	0.064	9
49 x 33	18	0.079	8
57 x 38	18	0.109	8
64 x 43	18	0.109	8
71 x 47	18	0.138	8
77 x 52	18	0.168	8
83 x 57	18	0.168	9

FILL HEIGHT TABLE 4.6

Corrugated Steel Pipe Arches, 3-inch by 1-inch Corrugations, Riveted, Welded, or Helical Fabrication, H-20 Loading

Pipe Dimensions Span x Rise (inches)	Minimum Cover, Top of Pipe to Top of Subgrade (inches)	Minimum Thickness Required (inches)	Maximum Fill Heights Above Top of Pipe (in ft.)
43 x 27	18	0.064	12
50 x 31	18	0.064	12
58 x 36	18	0.064	12
65 x 40	18	0.064	12
72 x 44	18	0.064	12
73 x 55	18	0.064	15+
81 x 59	18	0.079	15
87 x 63	18	0.079	14
95 x 67	18	0.109	13
103 x 71	24	0.109	12
112 x 75	24	0.109	11
117 x 79	24	0.109	10
128 x 83	24	0.138	9

FILL HEIGHT TABLE 4.7

Structural Steel Plate Pipe Arches, 6-inch by 2-inch Corrugations Bolted Fabrication, H-20 Loading

Pipe Dimensions Span x Rise (in. ft.)	Corner Radius (inches)	Minimum Cover, Top of Pipe to Top of Subgrade for 2 Tons per sq ft.(inches)	Minimum Thickness Required (inches)	Maximum Fill Heights Above Top of Pipe (in ft.)
6'1" x 4'7"	18	18	0.109	15
7'0" x 5'1"	18	18	0.109	15
7'11" x 5'7"	18	18	0.109	12
8'10" x 6'1"	18	24	0.109	11
9'9" x 6'7"	18	24	0.109	10
10'11" x 7'1"	18	24	0.109	9
11'10" x 7'7"	18	24	0.109	8
12'10" x 8'4"	18	24	0.109	8
14'1" x 8'9"	18	24	0.109	7
13'3" x 9'4"	31	24	0.109	13
14'2" x 9'10"	31	24	0.109	12
15'4" x 10'4"	31	24	0.138	11
16'3" x 10'10"	31	36	0.138	11
17'2" x 11'4"	31	36	0.138	10
18'1" x 11'10"	31	36	0.168	9
19'3" x 12'4"	31	36	0.168	9
19'11" x 12'10"	31	36	0.168	8
20'7" x 13'2"	31	36	0.188	8

Open Bottom Corrugated Steel Arches

The structural design of open bottom corrugated steel arches includes the determination of the required metal thickness and the footing design. These aspects of structural design will be discussed separately.

Steel Thickness Determination

The required steel thickness depends on the span length and height of fill over the arch. Table 4.8 gives the minimum and maximum fill heights for each combination of span and metal thickness for spans up to 25 feet. This table is based on the ring compression theory of determining fill heights as outlined in the "Handbook of Steel Drainage and Highway Construction Products" published by the American Iron and Steel Institute in 1971.

Open bottom arches with spans greater than 25 feet have not been standardized and, therefore, the manufacturers are required to do the steel design on an individual basis.

Footing Design

Every open bottom arch must be placed on concrete footings. The footings should be designed as ordinary spread footings. The following example design illustrates the design procedure.

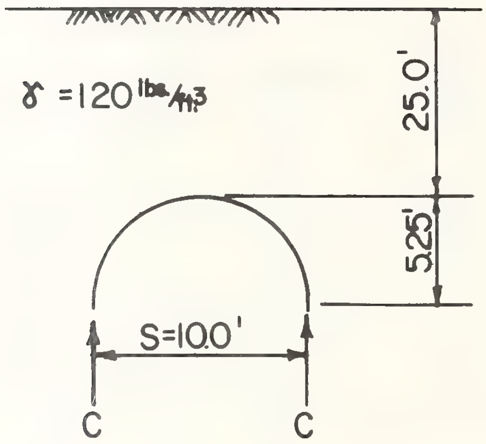
Loads

Dead Load

$DL = \gamma \times h$ $\qquad \gamma$ = soil density

$= 120 \times 25$ $\qquad$ h = height of fill

$= 3000\ {}^{lbs}/_{ft^3}$



Live Load

LL = 0 $\qquad$ From Figure 4.26

Note: Fill height makes the live load negligable.

Vertical Reaction

$C = P_v \times \frac{s}{2}$ $\qquad$ C = reaction forces

$P_v = K(DL+LL)$ $\qquad$ P_v = design pressure

$= .86(3000+0)$ $\qquad$ s = span

$= 2580\ {}^{lbs}/_{ft^2}$ $\qquad$ k = load factor from figure 4.27

$C = 2580 \times \frac{10}{2}$

$= 12{,}900\ {}^{lbs}/_{ft.}$

Note: The load on the footings will be something less than 12,900 lbs/ft. on the arch because the soil shearing forces on the arch are conservatively neglected. See "Reinforced Soil Bridge" by R.K.Watkins, Associate Director, Engineering Experiment Station, Utah State University, Logan, Utah.

Design footings for 12,900 lbs/ft. from arch.

Note: If the arch wall is not vertical at the footing it will be necessary to split C into the horizontal and vertical components.

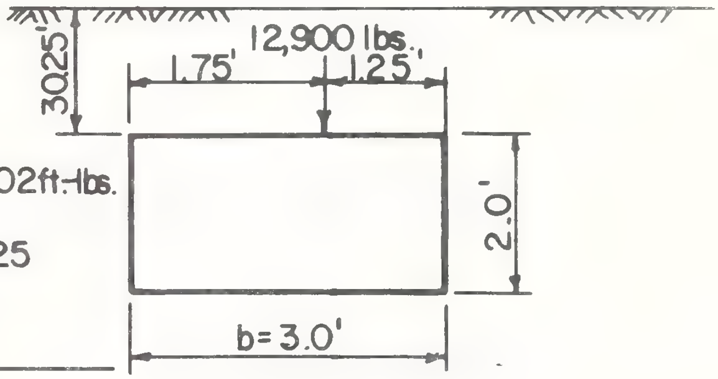
Dimensions

Allowable Soil Bearing Pressure

$p_a = 3 \ ^{tons}/_{ft^2}$ — p_a = allowable bearing pressure from core log and Tables 4.9 and 4.10.

Try a 3'-0" x 2'-0" deep footing

Assume 1.0' wide stripe (w=1.0')



Earth-(1.75)(30.25)(1.0)(120) = 6353 lbs. x -.63 = -4002 ft.-lbs.

Reaction- = 12,900 x .25 = 3225

Footing-(3.0)(2.0)(1.0)(150) = 900 x 0 = 0

R = 20,153 lbs. M = 777 ft.-lbs.

Footing Pressure (P)

$$P = \frac{R}{A} \pm \frac{MC}{I}$$

R = 20,153 lbs.

$$P = \frac{20,153}{3} + \frac{777 \times 1.5}{2.25}$$

$A = b \times w = 3 \times 1 = 3 \ ft^2$

$= 7236 \ ^{lbs}/_{ft^2}$

$= 3.6 \ ^{tons}/_{ft^2} > 3.0$

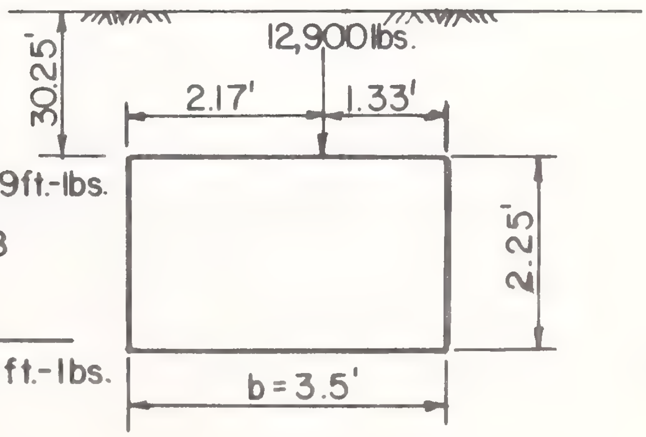
M = Moment about center of ftg. = 777 ft.-lbs.

$$C = \frac{b}{2} = \frac{3}{2} = 1.5 \ ft.$$

$P > p_a$ N.G.

$$I = \frac{wb^3}{12} = \frac{1.0(3.0)^3}{12} = 2.25 \ ft^4$$

Try a 3'-6" x 2'-3" deep footing



Earth-(2.17)(30.25)(1.0)(120) = 7877 lbs. x -.66 = -5199 ft.-lbs.

Reation- = 12,900 x .42 = 5418

Footing-(3.5)(2.25)(1.0)(150) = 1,181 x 0 = 0

R = 21,958 lbs. M = 219 ft.-lbs.

Footing Pressure (P)

$$P = \frac{R}{A} + \frac{MC}{I}$$

$$= \frac{21{,}958}{3.5} + \frac{219 \times 1.75}{3.57}$$

$$= 6381 \ ^{lbs}/_{ft^2}$$

$$= 3.19 \ ^{tons}/_{ft^2}$$

$P \simeq p_a$ O.K.

R = 21,958 lbs.

$A = 1.0 \times 3.5 = 3.5 \ ft^2$

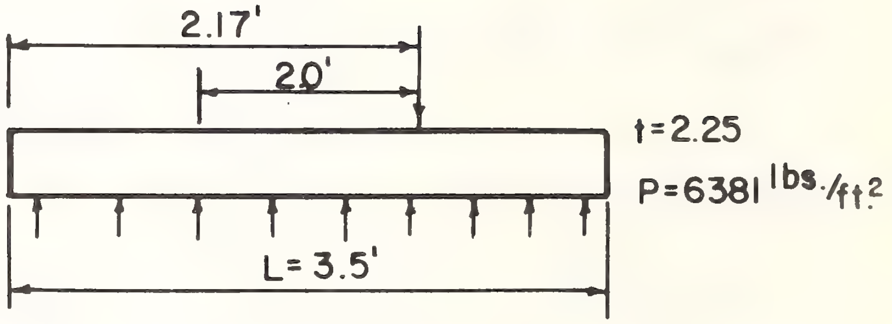
M = 219 ft.-lbs.

$C = \frac{b}{2} = 1.75$ ft.

$$I = \frac{wb^2}{12} = \frac{1 \times (3.5)^2}{12} = 3.57 \ ft^4$$

Use a 3'-6" x 2'-3" footing

Reinforcing Steel



$$M = \frac{P}{8}(L-a)^2$$ (page 8-70 Standard Handbook for Civil Eng.)

$$= \frac{6381}{8}(3.5-0)^2$$

$$= 9771 \text{ ft.-lb.}$$

$$\frac{6M}{t^2} \leq 1.6\sqrt{f'_c}$$ (page 8-70 Standard Handbook for Civil Eng.)

$$\frac{6 \times 9771}{(2.25 \times 12)^2} \leq 1.6\sqrt{3000}$$ f'_c = 28 day compressive strength of concrete

$$80.4 < 87.6$$

∴ No reinforcing is necessary

Place #4 bars @ 1'-0" centers longitudinally top and bottom as temperature steel.

Lab. Form 30
Revised 1-6-72

STATE OF MONTANA - DEPARTMENT OF HIGHWAYS
Helena, Montana

Core Log No. 6-542 Proj. No. S-410(5) Hole No. 9 Date Taken August 15,1973

Proj. Name Wampoo Creek Exploration Drilling for Soil Bearing Pressure

Location 14.0' Rt.C/L Sta.39+75 Driller Yost

Collar Elev. 2502.2 ft.
Depth of Hole 10.8'
Elev. of Ground Water 4.4'
Wt. of Hammer 140#
Average Fall of Hammer 30"

Depth	Legend	Description of Materials Type, Color & Consistency	Blow Counts	Penetration	
3.0		Medium dense brown sandy silty gravel-fill material	2" Split Spoon Penetration Test - 140# Hammer		
5.5		Brown sandy gravel			
7.5			6/7/8	1.5	1.0 to 2.5 No sample
		Loose brown sand w/ fine gravel	4/5/12	1.5	3.0 to 4.5
10.8		Very dense gray boulders and gravel	9/5/2	1.5	5.0 to 6.5
			9/50/R	0.9	7.0 to 7.9
			30/50/R	0.6	10.2 to 10.8
		Bottom hole same			
				7.9	Started Coring boulder with 5.0 ridged barrel with 6w/w diamond bit
			Drew	10.2'	100% recover
					10.0' of NX casing clean with glacier bit
					One box of samples

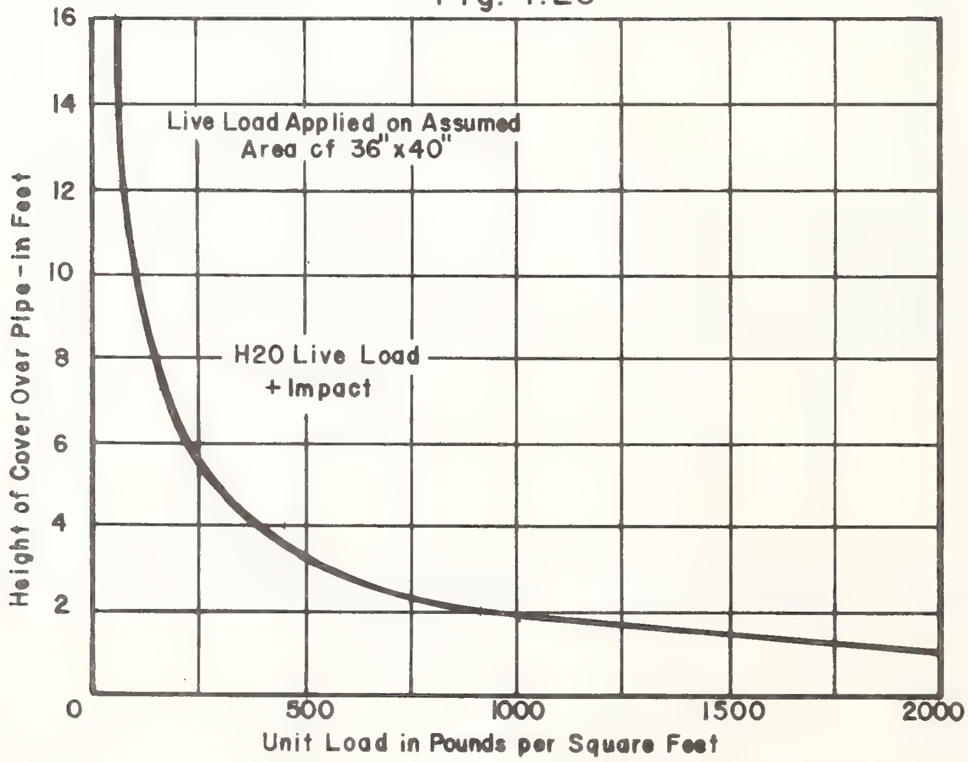
FILL HEIGHT TABLE 4.8

Open Bottom Corrugated Steel Arches, 6" x 2" Corrugations, H20 Loading

Span in Feet	Minimum Cover Inches	Maximum Cover in Feet — Specified Thickness in Inches						
		.109	.138	.168	.188	.218	.249	.280
5	12	40	60	78	87	102	117	132
6	12	34	50	65	73	97	104	110
7	12	29	43	56	62	73	84	94
8	12	25	37	49	55	64	73	82
9	24	22	33	43	48	57	65	73
10	24	20	30	39	44	51	58	66
11	24	18	27	35	40	46	53	60
12	24	17	25	32	36	42	48	55
13	24	15	23	30	33	39	45	51
14	24	14	21	28	31	36	42	47
15	24	13	20	26	29	34	39	44
16	24	12	18	24	27	32	36	41
17	36		17	23	25	30	34	38
18	36		16	21	23	27	31	35
19	36			19	21	25	29	32
20	36			18	20	23	27	30
21	36				18	21	24	28
22	36					20	22	25
23	36					18	20	23
24	36						19	21
25	48						17	19

Note: For spans over 25 feet, the manufacturer will be required to determine the required metal thickness.

Fig. 4.26



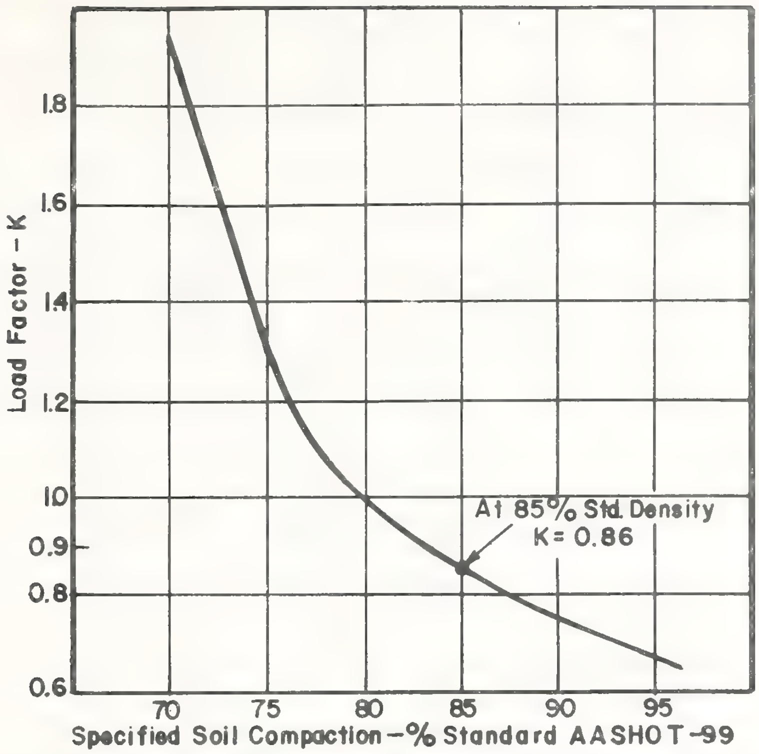


Fig.4.27—Load factors for corrugated steel pipe for backfill compacted to AASHO standard density. For example, at 85% the load factor K is 0.86%. This factor is applied to the total load to obtain the design pressure.

Table 4.9 Correlation of Standard-boring-spoon* Penetration with Soil Consistency and Strength

Soil Consistency	Number of blows per ft on spoon	Unconfined compressive strength, tons per sq ft.
Sand:		
Loose....................	15	
Medium compact...........	16-30	
Compact..................	30-50	
Very compact.............	Over 50	
Clay:		
Very soft................	3 or less	0.3 or less
Soft.....................	4-12	0.3-1.0
Stiff....................	12-35	1.0-4
Hard.....................	Over 35	4 or more

*2-inch spoon, 140-lb hammer, 30-inch fall.

Table 4.10 Allowable Bearing Values on Soils, Tons per Sq. Ft.

Massive crystalline bedrock: granite, gneiss, traprock--in sound condition	100
Foliated rock: schist and slate--in sound condition..................................	40
Sedimentary rock: hard shales, siltstones, sandstones--in sound condition..........	15
Exceptionally compacted gravels or sands..	10
Compact gravel or sand-gravel mixtures..	6
Loose gravel: compact coarse sand...	4
Loose coarse sand or sand-gravel mixtures; compact fine sand, or wet, confined coarse sand..	3
Loose fine sand or wet, confined fine sand..	2
Stiff clay..	4
Medium stiff clay...	2
Soft clay...	1

Reinforced Concrete Pipe

The "class" numbers are the principle measure of strength for reinforced concrete pipe. The required class depends on the fill height and type of bedding used. Table 4.11 gives the maximum and minimum fill heights for the various classes of pipe with Class C bedding and essentially no trench.

The minimum fill height is the minimum allowable distance between the top of the pipe and the finished roadway. However, the pipe should never be allowed to extend into the surfacing course.

Since the maximum fill height is normally not a problem with arch and elliptcal pipe, maximum fill height tables will not be given for these shapes. The maximum fill height will have to be calculated if necessary.

The minimum fill height for arch and elliptcal pipe can be determined from the minimum fill heights on Table 4.11. The minimum fill heights will be equal to the minimum height for the circular pipe whose outside diameter equals the outside span of the arch or horizontal elliptical pipe. Interpolation will normally be required since the span seldom exactly equals a listed diameter.

Fill Height Table 4.11
Round Reinforced Concrete Pipe, Class C. Bedding, H20 Live Load

Pipe Diameter (inches)	Pipe Class 2	3	4	5
12	8 (24")	11 (18")	16 (12")	24 (12")
15	8 (18")	11 (18")	16 (12")	25 (12")
18	8 (18")	11 (18")	17 (12")	25 (6")
21	8 (18")	12 (18")	17 (12")	25 (6")
24	9 (18")	12 (18")	17 (6")	26 (6")
27	9 (18")	12 (12")	18 (6")	26 (6")
30	9 (18")	12 (12")	18 (6")	26 (6")
33	9 (18")	12 (12")	18 (6")	26 (6")
36	9 (12")	12 (6")	18 (6")	27 (6")
42	9 (12")	12 (6")	18 (6")	27 (6")
48	9 (6")	13 (6")	18 (6")	27 (6")
54	10 (6")	13 (6")	18 (6")	27 (6")
60	10 (6")	13 (6")	19 (6")	27 (6")
66	10 (6")	13 (6")	19 (6")	27 (6")
72	11 (6")	13 (6")	19 (6")	27 (6")
78	11 (6")	13 (6")	19 (6")	
84	11 (6")	13 (6")	19 (6")	

Minimum Cover in Inches in Parentheses

4.15 DEBRIS CONTROL

The debris control data presented here is taken from Hydraulic Engineering Circular No. 9, entitled "Debris - Control Structures" published by the U.S. Department of Transportation, Federal Highway Administration.

Introduction

A frequent cause for the unsatisfactory performance or malfunction of highway drainage facilities is accumulation of debris at inlets. This accumulation may result in failure of the drainage structure and overtopping of the roadway by flood waters with ensuing damage to the embankment or to property upstream from the culvert. Debris control structures should be considered as an essential part of initial culvert design.

Debris can be controlled by three methods (a) intercepting the debris at or above the inlet; (b) deflecting the debris for detention near the inlet; and (c) passing the debris through the structure. In some locations, it may be desirable to provide a relief opening either in the culvert itself or by installing a separate smaller pipe higher in the embankment. The choice of method depends upon the size, quantity, and type of debris, the costs involved and maintenance proposed.

Often the waterway opening is arbitrarily increased in an attempt to pass debris through the curvert. Such an approach is usually more costly than installing a device to intercept debris. On the other hand when debris from the drainage basin is passed through the structure without clogging, maintenance costs are much less than when debris is intercepted and subsequently removed. Depressed or sloping inlets can be used to reduce ponding and accelerate velocities thus preventing deposition of debris at the culvert inlet.

When clogging of a drainage structure by debris is a problem, a debris-control structure may have several of the following advantages:

(a) Prevents traffic delays due to accumulation of drift on the roadway or washouts caused by clogged culverts.

(b) Allows for planned maintenance rather than emergency maintenance during floods when other situations arise which also require immediate attention.

(c) Avoids providing a "safety factor" in sizing a culvert to accommodate an estimated quantity of debris.

(d) Provides a safeguard against damaging buoyant forces when an accumulation of drift at the culvert entrance causes partfull flow.

(e) Gives maintenance forces a remedial method for correcting a drift problem.

The types of debris-control structures are listed below followed by a system for classifying the type of debris expected from the drainage basin. The basis for choosing the type of control is given and details of design are then discussed.

Types of Debris-Control Structures

Debris-control structures can have many shapes and can be constructed of a variety of materials. For discussion here these structures will be divided into the following general types:

1. Debris Deflectors - (Photos 1 - 13) - Structures placed at the culvert inlet to deflect the major portion of the debris away from the culvert entrance. They are normally "V"-shaped in plan with the apex upstream.
2. Debris Racks - (Photos 14 - 27) - Structures placed across the stream channel to collect the debris before it reaches the culvert entrance. Debris racks are usually vertical and at right angles to the streamflow, but they may be skewed with the flow or inclined with the vertical.
3. Debris Risers - (Photos 28 - 34) - A closed-type structure placed vertically over the culvert inlet, to cause deposition of flowing debris

and fine detritus before it reaches the culvert inlet. Risers are usually built of metal pipe.

4. Debris Cribs - (Photos 35 - 39) - Open crib-type structures placed vertically over the culvert inlet in log-cabin fashion to prevent inflow of coarse bedload and light floating debris.
5. Debris Fins - (Photos 40 - 45) - Walls built in the stream channel upstream of the culvert. Their purpose is to align debris, such as logs, with the axis of the culvert so that the debris will pass through the culvert barrel without clogging the inlet. They are sometimes used on bridge piers for deflecting drift.
6. Debris Dams and Basins - (Photos 46 - 50) - Structures placed across well defined channels to form basins which impede the stream flow and provide storage space for detritus and debris.
7. Floating Drift Boom - Logs or timbers which float on the water surface to collect floating drift. Drift booms require guides or stays to hold them in place laterally. They are limited in use and will not be discussed further.

Classification of Debris

Flood flow reaching a culvert nearly always carries debris which may be either floating material, material heavier than water, or a combination of both. Debris concerns the highway engineer because it can be deposited at the culvert entrance or in the culvert, thus impairing its operation. A thorough study of the extent and type of the debris orginating in the drainage basin is essential for proper design of a culvert.

As an aid in selecting the proper debris-control structure, the debris from the drainage basin should be classified. A convenient classification system is that of the California Division of Highways which follows:

1. Very Light Floating Debris or No Debris
2. Light Floating Debris - Small limbs or sticks, orchard prunings,

and refuse.

3. Medium Floating Debris - Limbs or large sticks.
4. Heavy Floating Debris - Logs or trees.
5. Flowing Debris - Heterogeneous fluid mass of clay, silt, sand, gravel, rock, refuse or sticks.
6. Fine Detritus - Fairly uniform bedload of silt, sand, gravel more or less devoid of floating debris, tending to deposit upon diminution of velocity.
7. Coarse Detritus - Coarse gravel or rock fragments.
8. Boulders - Large boulders and large rock fragments carried as a bedload at flood stage.

DESIGN OF DEBRIS-CONTROL STRUCTURES

Preliminary Field Studies

Proper design of a debris-control structure must be preceded by a field study of the debris problem. Among the factors to be considered are possible future changes in the type of debris that might result from new industry or changes in land use within the drainage basin. As an example, logging in a previously virgin area could change the nature of the debris problem from one of "medium floating debris" to "heavy floating debris". Fire also could change the type and quantity of debris reaching culverts making it necessary to take remedial action for debris control.

Culverts located at the end of urban drainage channels are often clogged by trash dumped into the channel or washed off the city streets. Under such conditions, a rack can usually be installed at low cost to prevent clogging. However, urban locations require careful design since malfunction of the debris-control structure will often cause flooding and damage adjacent property.

An estimate of the quantity as well as the type of debris is needed by the designer so that adequate debris storage can be provided. Information on the

types and quantities of debris resulting from past floods can be an invaluable guide in selection of the type of debris-control structure. Such information could be secured from maintenance personnel, from inhabitants of the immediate area or by personal observation. Access to the debris storage area is needed for cleaning.

A knowledge of allowable headwater and the height of embankment above the invert elevation at the culvert inlet is also necessary in determining the type of structure best suited to the particular problem. Damage that would result from a plugged culvert should be estimated to evaluate the need for a debris-control structure.

To summarize, the field survey data should include:

1. Classification as to type of expected debris.
2. Quantity of expected debris.
3. Future changes in debris type or quantity due to proposed land use.
4. Estimate of streamflow velocities in the vicinity of the culvert.
5. Storage available for debris at the site, access to the structure for maintenance and the standard of maintenance planned.
6. Possible damage that would result from debris clogging the drainage structure without protection.

Materials and Construction

Among the materials that have been used successfully for debris-control structures are:

1. Railroad rail
2. Structural steel sections, such as I-beams
3. Timber
4. Precast concrete
5. Cable
6. Culvert pipe
7. Fencing Material

Each of these materials has advantages and disadvantages depending on site conditions and type of debris-control structure. Railroad rail has been used, but it has been found that structural steel sections are comparable in cost and are somewhat easier to fabricate and handle. Some maintenance personnel prefer bolted connections to welded connections because they are easier to repair.

Several modifications can be made to adapt a design to a given situation. For example, Photo 19 shows a roadside wall used with a debris rack to prevent flow of debris onto the roadway. A relief opening can be supplied for little additional cost in those locations where risk of clogging of a debris structure or unprotected culvert is great (Photo 33).

Field surveys have indicated that the foundation of debris-control structures is subject to erosion. The debris fin is a type of structure for which special emphasis must be placed on foundation treatment (Photo 42).

Selecting Type of Structure

Most debris-control structures will intercept debris, however, not all structures are equally efficient. Table 4.12, based on experience with different types of structures, provides a guide for selecting suitable types of structures for various debris classification.

TABLE 4.12 Guide for selecting type of structures suitable for various debris classifications

Debris Classification \ Type of Structure	Deflector	Rack	Riser	Crib	Fin	Dam and Basin	Boom	Sloping Inlet
Light Floating Debris		X		X			X	
Medium Floating Debris	X	X					X	
Heavy Floating Debris	X				X			
Flowing Debris			X			X		X
Fine Detritus			X			X		X
Coarse Detritus			X	X		X		X
Boulders	X							

Debris Deflectors

The function of a debris deflector (Photos 1 - 13) is to cause debris to be diverted from the culvert inlet and to accumulate in a storage area where it can be removed after the flood subsides. The storage area provided must be adequate to retain the estimated type and quantity of debris accumulated during any one storm or between removal. The deflector should be built at the culvert entrance. If it is placed some distance upstream, debris may return to the channel between the deflector and the culvert inlet. The deflector requires little maintenance and reduces the collection of debris in front of the culvert inlet.

Debris deflectors are usually built of heavy rail or steel sections (Photos 1 - 11) although timber (Photos 12, 13) and steel pipe are sometimes used for light debris. Single deflectors can be built over batteries of pipe culverts (Photo 6) or individual deflectors can be built over each pipe of a battery (Photo 11). Their structural stability and orientation with the flow make deflectors particularly suitable for large culverts and high velocity flow. They are most suitable for heavy logs, stumps, large boulders, or rocks (Photo 1).

Photo 10 and Plate II show a deflector that uses a cable as its lower longitudinal member. This modification has proved superior in locations where heavy boulders damage rigid members. Wire and post debris deflectors (Photo 9) have been used for light floating debris.

Plates I and II show general dimensional details of debris deflectors. The angle at the apex of the deflector should be between 15° and 25° and the total area of the two sides of the deflector should be at least 10 times the cross sectional area of the culvert. Spacing between vertical members should be no larger than the minimum culvert dimension and no smaller than 1/2 the minimum dimension. A spacing of 2/3 the minimum dimension is generally used. The base width and height of the deflector should be at least 1.1 times the respective dimensions of the culvert. Where the design flood is above the top elevation of the deflector and floating debris is anticipated, horizontal members should be placed across the top. The spacing of horizontal members on the top should be no greater than 1/2 the smallest dimension of the culvert opening. The upstream member is vertical on most existing installations. However, a sloping member at the apex (sloping downstream from bottom of member) would reduce the impact of heavy floating debris and boulders, and probably prevent debris from gathering at that point. Deflectors with a sloping member at the apex are highly recommended by maintenance personnel. The debris deflector should be aligned with the stream rather than the culvert as shown in Photo 2 or debris will tend to block the channel.

Debris Racks

A debris rack (Photos 14-27) is essentially a barrier across the stream channel which stops debris that is too large to pass through the culvert. Debris racks vary greatly in size and in the material used in their construction. Height of racks should allow some freeboard above the normal depth of flow in the upstream channel for the design flood. Racks 10 to 20 feet high have been constructed. The

rack may be vertical or inclined and may be placed over the culvert inlet (Photos 14, 15, 19, 21, 22, 23, 24, 26, 27, and 29) or upstream from the culvert (Photos 16, 18, and 25). Racks should not be placed in the plane of the culvert entrance, since they induce plugging when thus positioned. A means of access to the rack is necessary for maintenance.

The rack should be placed well upstream from the culvert entrance in those locations where a well defined channel exists. However, they should not be placed so far upstream that debris enters the channel between the rack and the culvert inlet. If a large debris storage area exists at the rack location, the frequency of maintenance is reduced and added safety is provided against overtopping the installation during a single storm. Some racks have not required maintenance for several years.

Plates III through VI inclusive show the general dimensional details of debris racks. The total straining area of a rack should be at least ten times the cross sectional area of the culvert being protected. Vertical bars are generally spaced from 1/2 to 2/3 the minimum culvert dimension. This spacing permits the lighter debris to pass through the rack and the culvert. In urban areas, bar spacing of racks should be a maximum of 6 inches and tied to the culvert headwall by top bars to prevent entrance by children.

Generally, racks do not have top or horizontal members extending from the rack to the culvert headwall although there are exceptions (Photo 15). The overall dimensions of the rack should be a function of the amount of debris expected per storm, the frequency of storms, and the schedule of expected clean-outs. When a rack is installed at the upstream end of the wingwalls, it should be at least as high as the culvert parapet.

Vertical racks receive the full impact of floating debris and boulders. Inclined racks and rubber tires (Photos 17) have been used to help reduce the impact of heavy debris striking at high velocity.

Chain-link fence has been used for removal of light debris where stream velocities are low. Particular advantage has been noted in tidal areas where the functioning of flap or check gates which prevent back flow from rising and falling tides is negated unless light debris is kept from gathering on gate seats and thereby blocking closure of the gates.

Debris Risers

Debris risers (Photos 28 - 34) generally consist of a vertical culvert pipe and are usually suitable for culvert installations of less than 54 inch diameter. This type of debris-control structure is used where considerable height of embankment is available and where debris consists of flowing masses of clay, silt, sand, sticks, or medium floating debris without boulders. Risers are seldom structurally stable under high-velocity flow conditions because of their vulnerability to damage by impact.

Risers placed above the streambed at the bottom of steep, narrow draws cause ponding with a subsequent reduction in velocity. Deposition of sediment follows as a consequence of the reduced velocity. The resulting flat-bottom basin gives maintenance personnel a place to work when either culvert clean-out or debris removal is necessary. This basin also causes deposition of heavier debris upstream at the entrance to the basin where the debris cannot clog the drainage structure. To avoid vibration of the riser pipe and unstable flow conditions, the riser diameter should be about 1-foot larger than the culvert diameter.

Plates VII - X inclusive show the general dimensional details of debris risers. The riser should be covered by a grate or cage to prevent clogging of the culvert. The grate bars can be reinforcing steel or other such material with vertical spacing not greater than 1/2 the diameter of the culvert. Slots or holes are placed in the sides of the riser to carry low flow (Photo 32). It is preferabl to have these holes punched before galvanizing to avoid deterioration by rust. The

holes are considered to have no hydraulic capacity under peak flow conditions because of the likelihood of their becoming plugged by light floating debris and silt. It is good practice to build riser pipes at least 36 inches in diameter to provide an area large enough for maintenance access. It is also desirable to connect the grate bars to a coupling band, rather than directly to the riser pipe, so the grate can be removed should cleaning be required. If the embankment is of sufficient height, provisions should be made to extend the riser vertically if necessary. This can be accomplished by means of standard coupling bands in the case of corrugated metal pipe risers.

Debris Cribs

A debris crib (Photos 35 - 39), often called a "bear trap", is particularly adapted to small-size culverts where a sharp change in stream grade or constriction of the channel causes deposition of detritus at the culvert inlet. The crib is usually placed directly over the culvert inlet and is generally built up in log-cabin fashion although other designs are sometimes used.

Plate XI shows the general dimensional details of a debris crib. Spacing between bars should be about 6 inches. A crib may be open (Photos 36 - 38) or covered (Photos 35, 39) with horizontal top members spaced equal to the crib members. Debris can almost envelope a crib without completely blocking the flow and plugging the culvert. When an open crib is used as a riser and an accumulation of detritus is expected to build up, provision can be made for increasing the heights as needed (Photo 36, 37). Cribs and risers are somewhat similar, but cribs are more appropriate than risers where the culvert has little cover and the detritus is coarse. Cribs have been built as high as 50 feet above a pipe invert with little change in the efficiency of the facility. Due to the debris type and site conditions associated with debris risers and cribs, field inspections of all types of existing debris-control structures have shown these two types to be most consistently successful in producing an efficient maintenance free installation.

Debris Fins

Debris fins (Photo 40 - 45) have been used successfully with large culverts where the debris consists of logs or other material that would pass through the pipe if oriented with their long dimension parallel to the flow. The purpose of a fin is to align debris so that it will pass through a culvert.

The debris fin is a thin wall installed parallel to the flow. It is usually concrete and located on the center line of a single culvert (Photos 43 - 45) or as an extension of the interior walls of a multiple-box culvert (Photo 40 - 42). The upstream edge of the fin should be rounded and sloped toward the culvert to reduce impact, turbulence, and the probability of gathering debris as shown in Photos 40 - 41 rather than in a vertical plane as shown in Photos 42 - 45.

A debris fin is usually constructed to the height of the culvert; hence, its effectiveness is limited after the inlet becomes submerged. Based on experience a fin length of 1.5 to 2 times the height is recommended. The leading edge would thus have a slope of from 1.5:1 to 2:1. The wall thickness should be a minimum to satisfy structural requirements, thus offering little disturbance to flow. Fins are generally not used on culverts with a minimum dimension less than 4 feet. Debris fins are subject to the same erosive forces as bridge piers. Therefore, care <u>must</u> be taken in the design of the footings. Similar fins constructed on bridge piers reduce the collection of drift.

Debris Dams and Basins

In streams carrying heavy sediment load it is usually impossible to pass the bulked flow through the culvert. If the height of embankment is not sufficient for a riser or crib, a debris dam and settling basin placed some distance upstream from the culvert might be feasible. Debris dams and silting basins are sometimes used to trap heavy boulders or coarse gravel tending to clog culverts, especially on low fills. In some locations debris dams have been built to provide

the added benefit of ground water recharge resulting from ponded water.

Debris dams (Photos 46 - 51) can be built of precast concrete beams placed in crisscross or log-cabin fashion with rock dumped between the members (Photos 50). Other dams have been built of rock held in place by wire (Photos 47, 49).

The extent of preliminary investigation required for the design of a dam should be commensurate with the size and cost of the structure. Information is needed concerning watertightness of the reservoir, suitability of the foundations for supporting the dam, and the availability of construction materials.

Earth or rock fill dams are usually desirable. A spillway constructed as a channel outside the limits of the dam is preferred. A number of debris dams were built in Southern California and were found to have lower construction costs than the annual cost of removing the debris that otherwise would have been deposited adjacent to and within drainage structures.

Combined Debris-Controls

Each drainage basin presents its own debris problem. Often more than one problem exists and two or more types of debris-control structures must be used. At some locations it may be preferable to remove the heavy debris at a location upstream from the culvert and to remove the fine material nearer the culvert inlet. At other locations it may be advisable to install two types of devices so that one will function if the other fails. For example Photo 33 shows a debris riser installed over the entrance of a culvert to provide the water access to the culvert in the event the culvert entrance becomes plugged. Photo 34 shows the same installation after a flood.

Photos 43 and 45 show culverts protected by both a debris fin and a debris riser. Photo 51 shows an installation consisting of a debris dam and settling basin, with a culvert and debris deflector at its base followed by a debris

rack at the culvert inlet in the background.

A sloping inlet protected by a debris rack, such as that shown in Photo 14, might be considered a combined debris-control structure. In addition to increasing the hydraulic capacity, a sloping inlet induces velocities high enough to prevent fine material settling out at the inlet. The rack will divert the debris too large to pass through the culvert.

Maintenance

The standard or frequency of maintenance must be considered in the design of a debris-control structure. Structures located on a primary highway may have a higher frequency of maintenance than those on a secondary highway. If a low standard of maintenance is to be provided, it may be desirable to use a different type debris-control structure requiring less attention or choose a larger culvert. This consideration may also determine the best of two or more alternatives.

Provisions must be made for maintenance access to the debris-control structure site. A means of access is often difficult to provide, particularly where a high embankment exists. However, such installation usually require less maintenance because of the added debris storage area available. If haul roads to debris-control installations are not practical, it may be necessary to provide an area where mechanical equipment such as a crane could be located for removing debris without disrupting highway traffic. Some debris barriers must be cleaned after each storm. Occasionally accumulated debris might be fed piece by piece through the drainage structure.

Maintenance problems may require modifications in design. For example, debris clogging an extremely long culvert is difficult to remove. In such cases positive debris control could become essential and the size of openings in the debris-control structure reduced to remove all the debris from the flow entering the culvert (Photo 22).

Photo 1. Steel rail debris deflector for large rock.

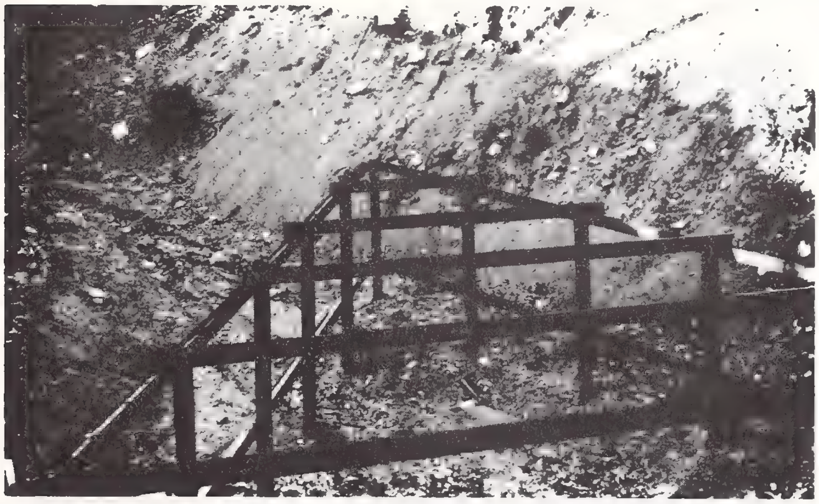


Photo 2. Steel rail debris deflector installed with major axis parallel to culvert rather than stream.

Photo 3. Steel rail debris deflector for fine detritus.

Photo 4. Steel rail debris deflector in area of heavy flowing debris.

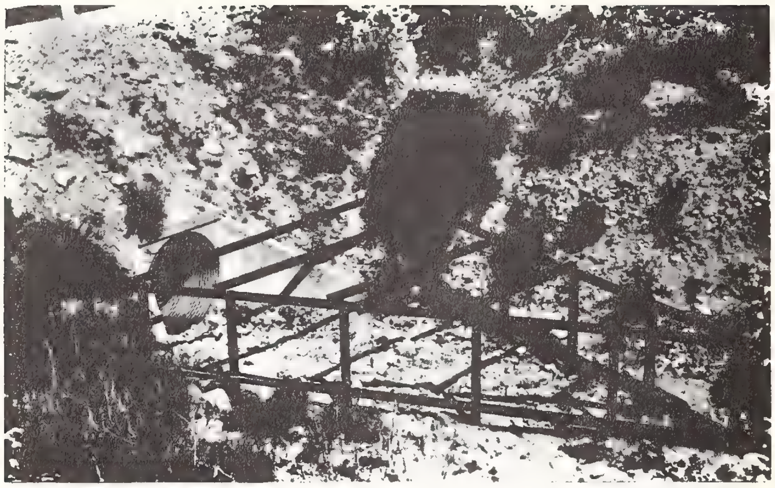


Photo 5. Steel rail debris deflector.

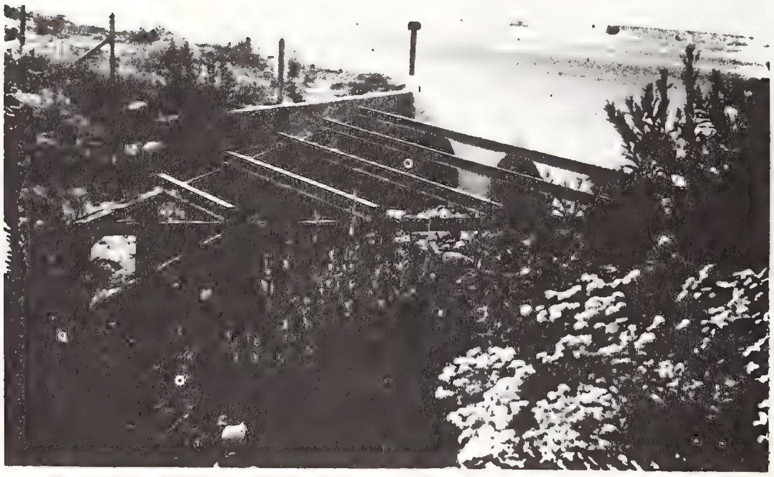


Photo 6. Steel rail debris deflector for battery of culverts (See Photo 7.)

Photo 7. Installation shown in Photo 6. during flood; function well under heavy debris flow.

Photo 8. Steel rail debris deflector. Note storage area for debris resulting from culvert projection.

Photo 9. Wire and post debris deflector for light floating debris.

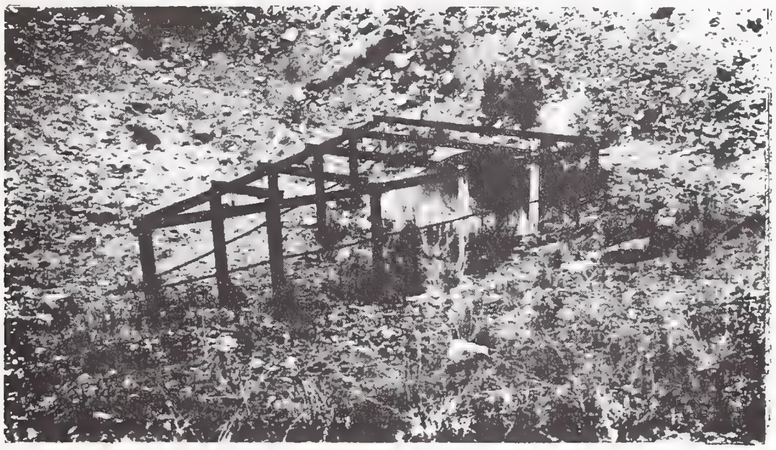


Photo 10. Steel rail and cable debris deflector. Cable's flexibility more desirable than rail's rigidity in boulder areas.

Photo 11. Steel debris deflectors installed at entrances to a battery of culverts.

Photo 12. Timber pile debris deflector for boulders and heavy floating debris.

Photo 13. Timber pile debris deflector protected culvert during heavy floods. Nearby culverts without deflectors were plugged.

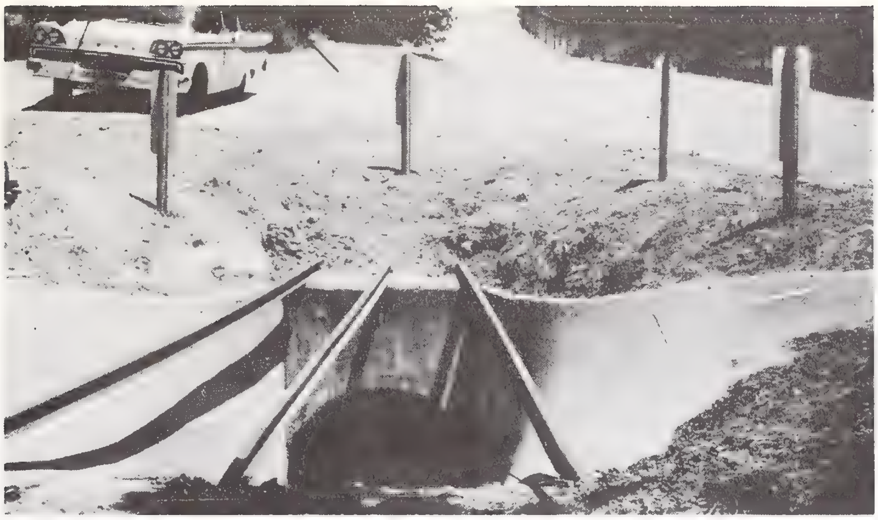


Photo 14. Rail debris rack over sloping inlet. Heavy debris and boulders ride over rack and leave flow to culvert unimpeded.

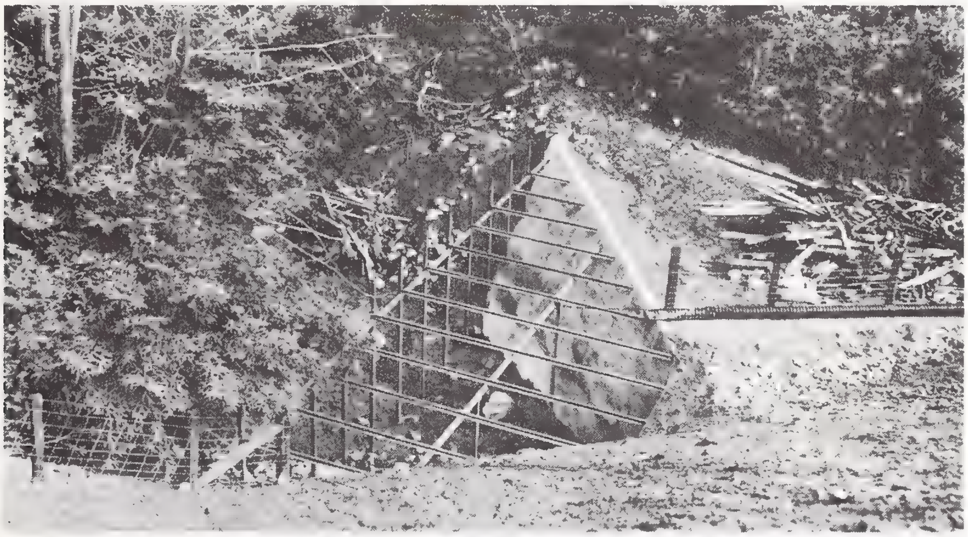


Photo 15. Rail debris rack with top members in area of logging operations.

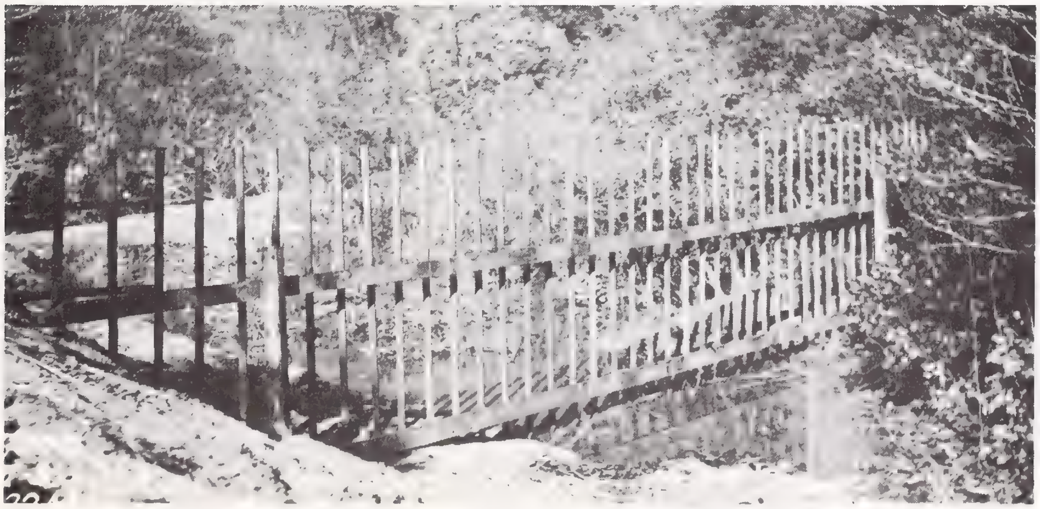


Photo 16. Post and rail debris rack, in place for 35 years, for light to medium floating debris installed 100' upstream of culvert.

Photo 17. Steel debris rack downstream of culvert on beach. Rubber tires reduce impact of logs.

Photo 18. Rail debris rack. Note large straining area provided.

Photo 19. Steel debris rack hinged to facilitate maintenance of drop inlet. Low wall to impede passage of debris onto roadway.

Photo 20. Debris control hinged installation of reinforcing steel at inlet to roadside downdrain.

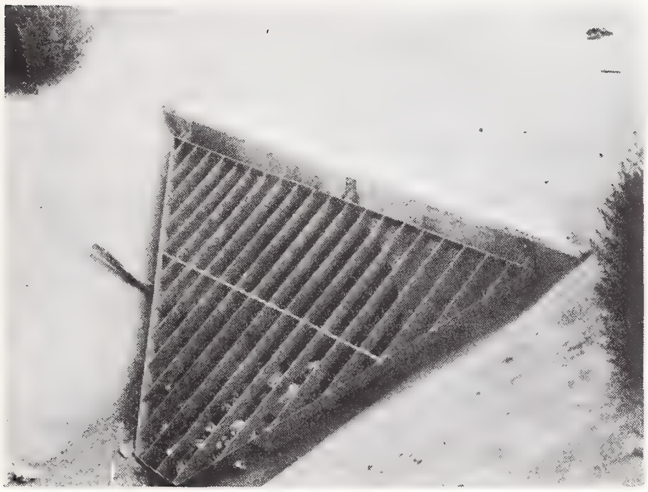


Photo 21. Hinged steel debris rack in urban area. Due to nature of debris and possible entry by children, bar spacing is close.

Photo 22. Debris rack used in State of Washington. (See Plate III for design dimensions.)

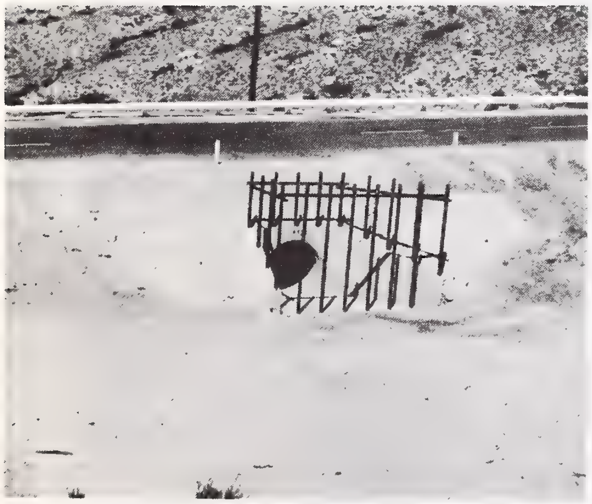


Photo 23. Rail debris rack in arid region. (See Photo 24.)

Photo 24. Installation shown in Photo 23. after several years of fine silt deposition at entrance.

Photo 25. Steel debris rack probably saved the culvert from plugging.

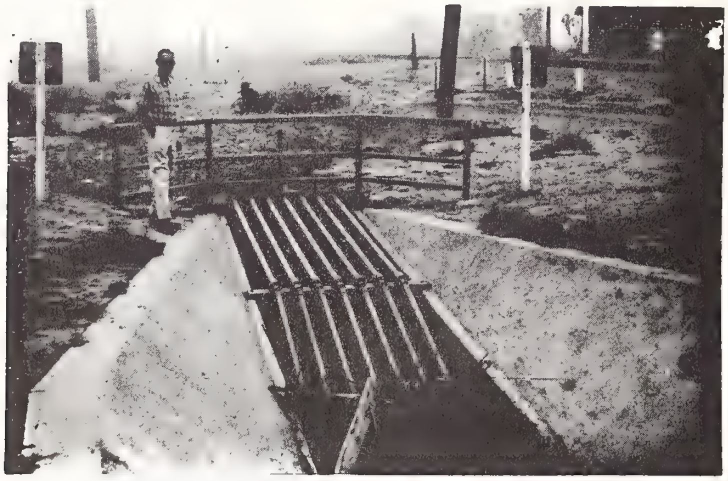


Photo 26. Pipe grill debris rack. Vertical fence at downstream end to prevent debris from spreading over ponding area.

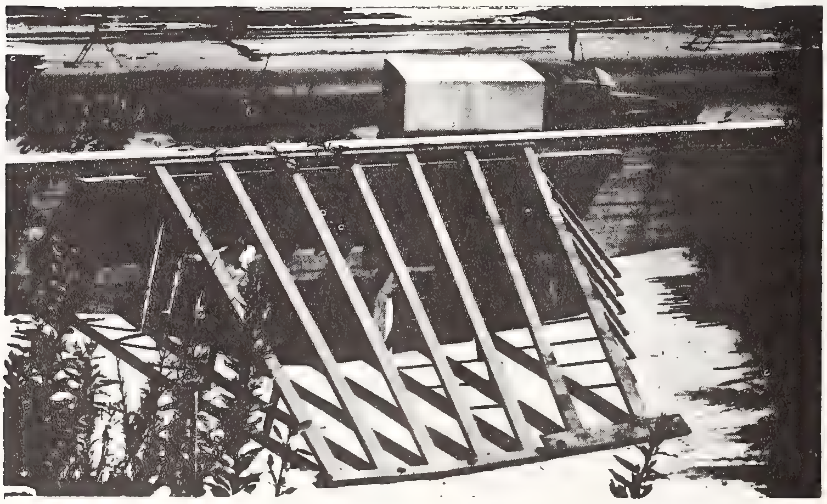


Photo 27. Steel grill debris rack with provision for cleanout afforded by concrete paved area in foreground.

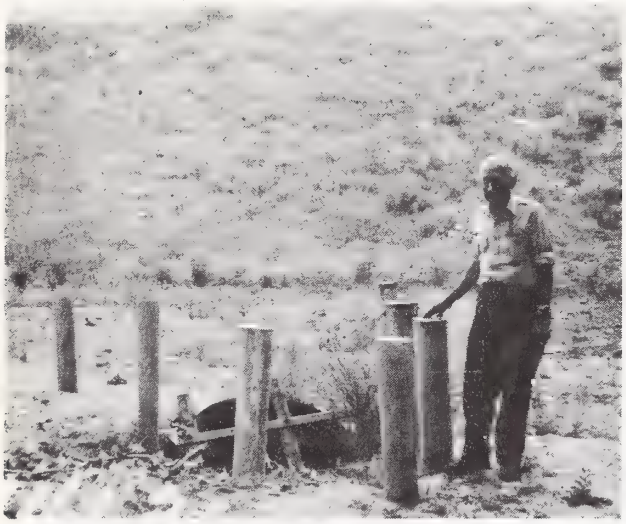


Photo 28. Metal pipe debris riser, with posts to deflect boulders, installed by maintenance forces on 45° angle to vertical.

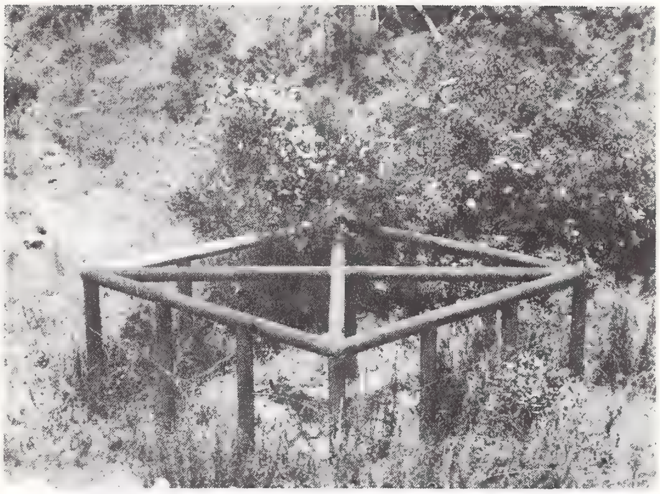


Photo 29. Post debris rack placed over entrance to metal pipe debris riser after latter had caused deposition.

Photo 30. Metal pipe debris riser required little maintenance. Basin had built up 10'.

Photo 31. Metal pipe debris riser, in place for 25 years, operated well without vertical extension.

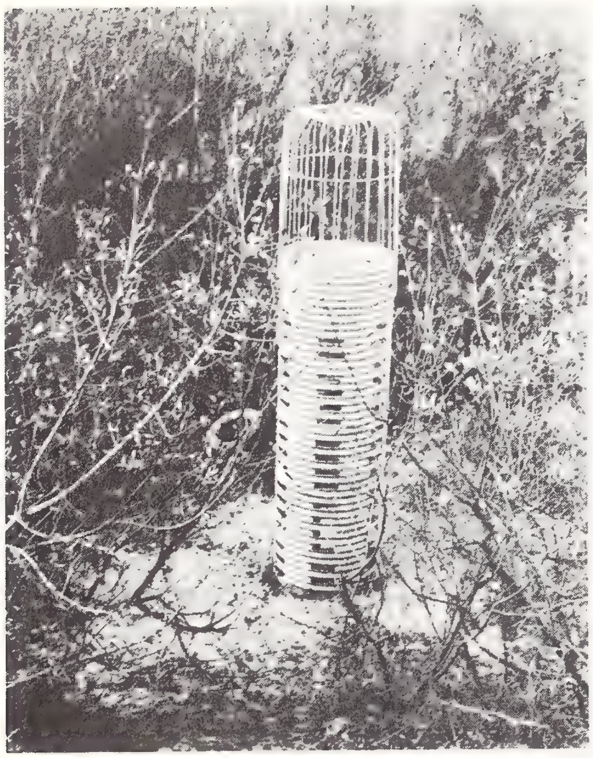


Photo 32. Metal pipe debris riser shows slots for low flows.

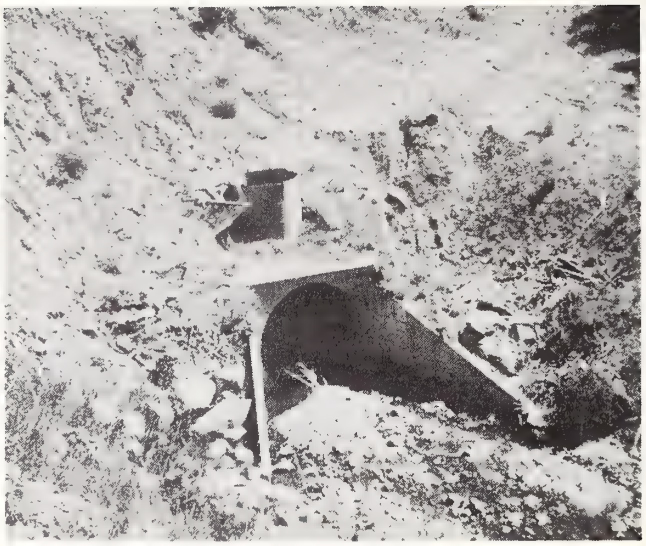


Photo 33. Metal pipe debris riser placed during initial construction of culvert provides relief in case the latter becomes plugged. (See Photo 34.)

Photo 34. Installation shown in Photo 33. after flood. Riser carried heavy flow during flood. Fence partially surrounding riser of no value for debris control. (Note man at center of photograph.)

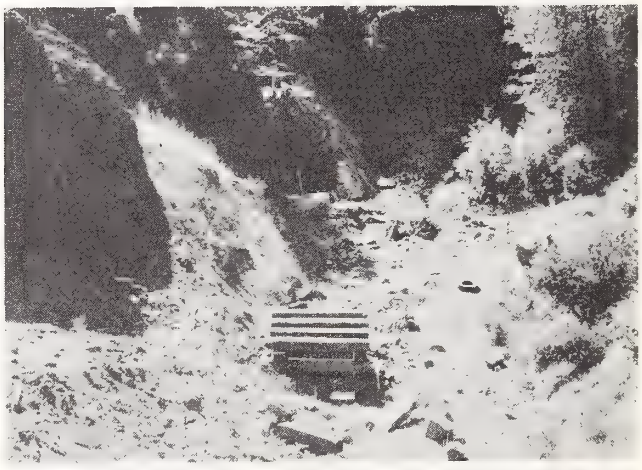


Photo 35. Timber debris crib in ideal location, i.e., high roadway embankment and large settling basin.

Photo 36. Debris crib of precast concrete sections and metal dowels. Height increased by extending dowels and adding more sections.

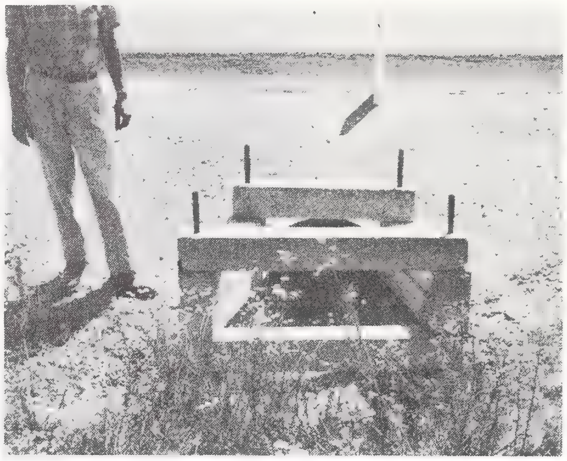


Photo 37. Debris crib of precast concrete sections and metal dowels.

Photo 38. Timber debris crib of inexpensive local materials.

Photo 39. Redwood debris crib with spacing to prevent passage of fine material. Basin had built up 30'.

Photo 40. Concrete debris fins with sloping leading edges as extensions of culvert walls.

Photo 41. Concrete debris fin with sloping leading edge as extension of center wall.

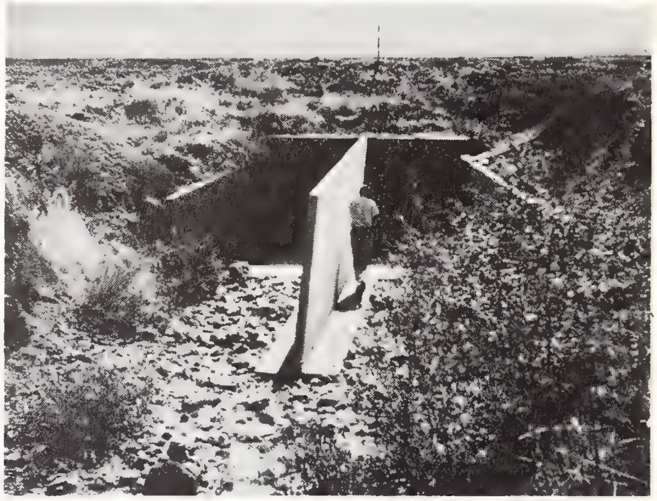


Photo 42. Concrete debris fin with rounded vertical leading edge as extension of culvert center wall.

Photo 43. Concrete debris fin and metal pipe debris riser in conjunction with single corrugated metal pipe culvert.

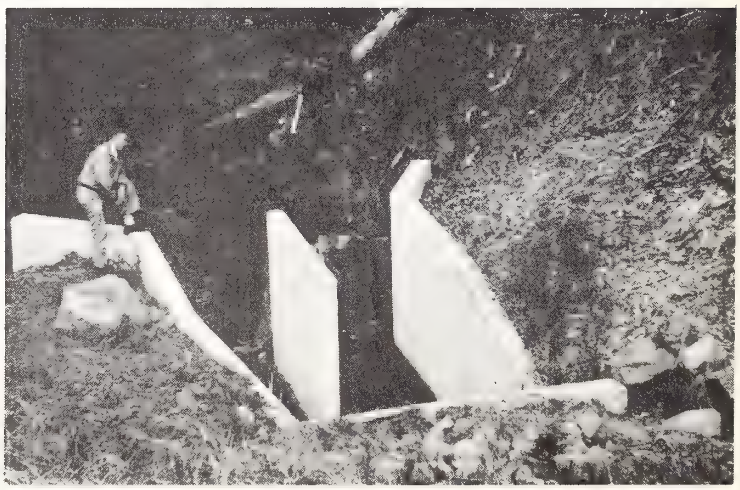


Photo 44. Concrete debris fin for single culvert. Preferable if more area existed between wingwalls and fin.

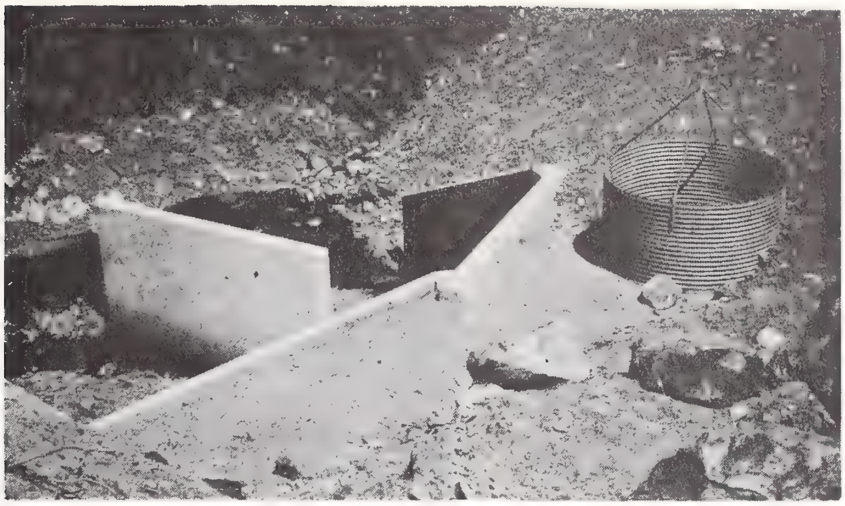


Photo 45. Debris fin and metal pipe debris riser in conjunction with single barrel culvert.

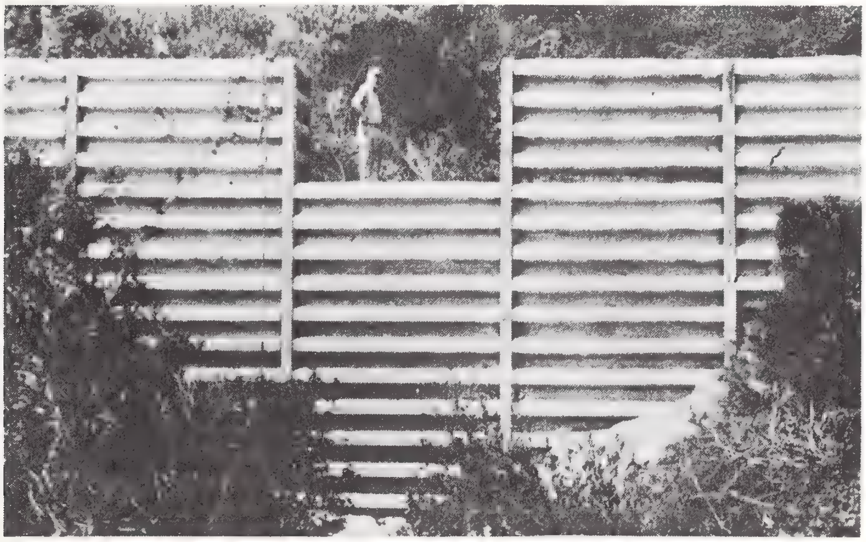


Photo 46. Metal bin type debris dam.

Photo 47. Debris dam of rock and wire.

Photo 48. Debris dam and basin in foreground and steel grill debris rack at culvert entrance in background. (See Photo 49.)

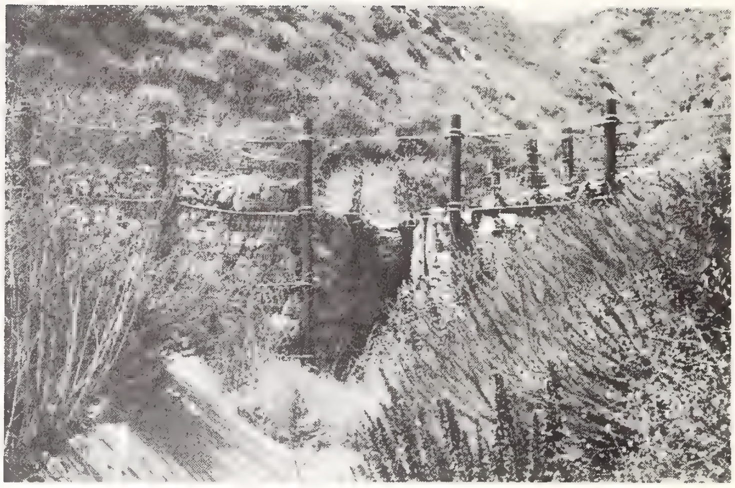
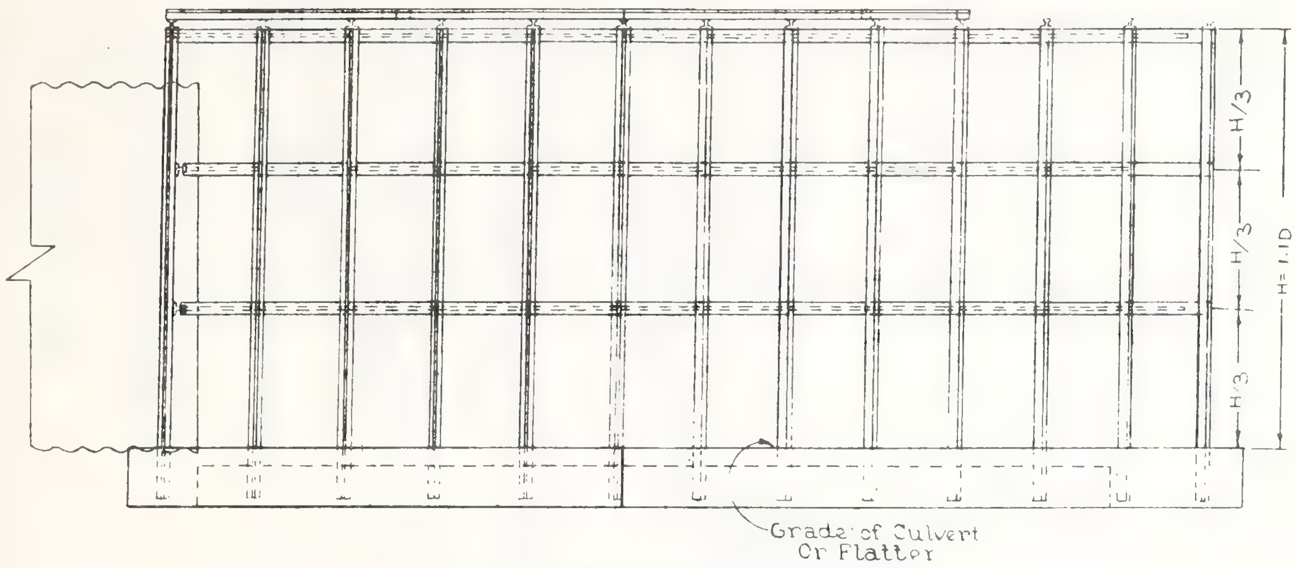


Photo 49. Debris dam of rock and wire shown in Photo 48.

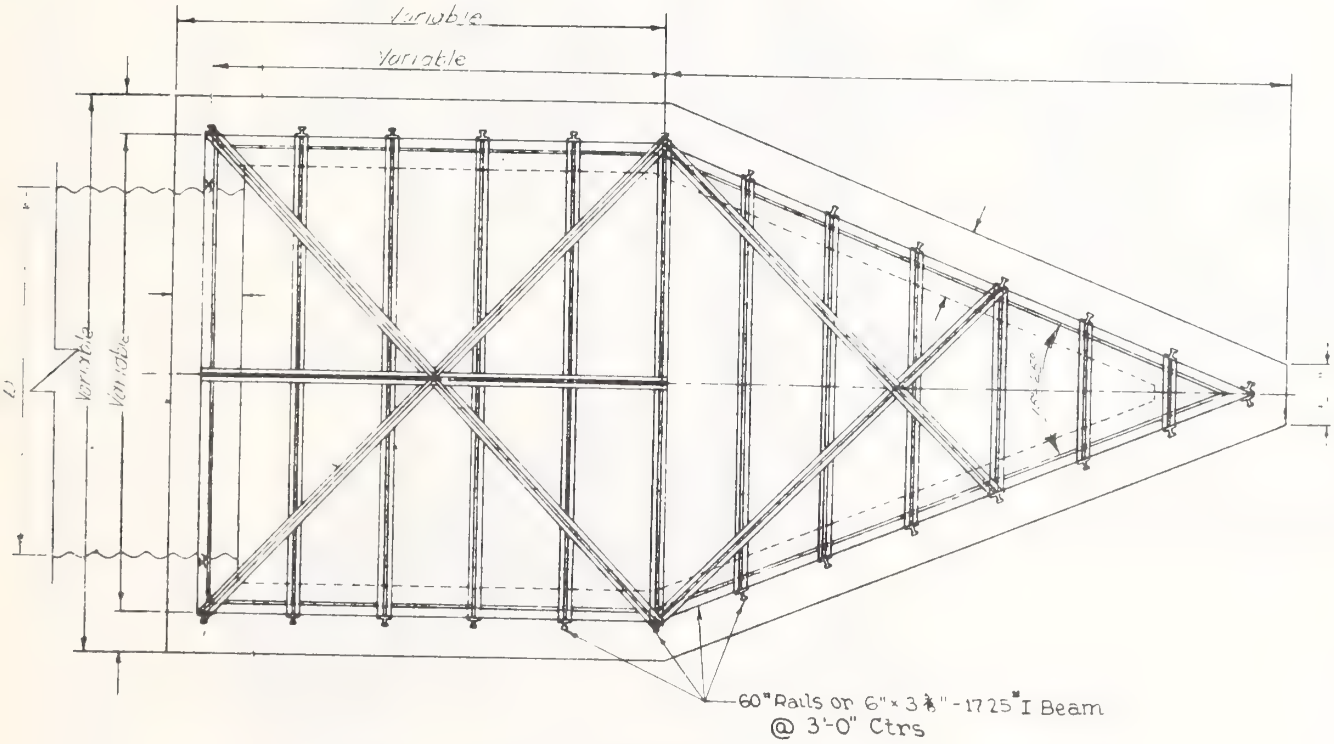
Photo 50. Debris dam of precast concrete sections fabricated to enable placement in interlocking fashion.

Photo 51. Debris dam and basin along with steel debris rack over culvert entrance in foreground. Metal pipe riser visible over the spillway. Roadway in background.

PLATE I



SIDE ELEVATION



PLAN

DEBRIS DEFLECTOR

CALIFORNIA DIVISION OF HIGHWAYS

DIVISION II

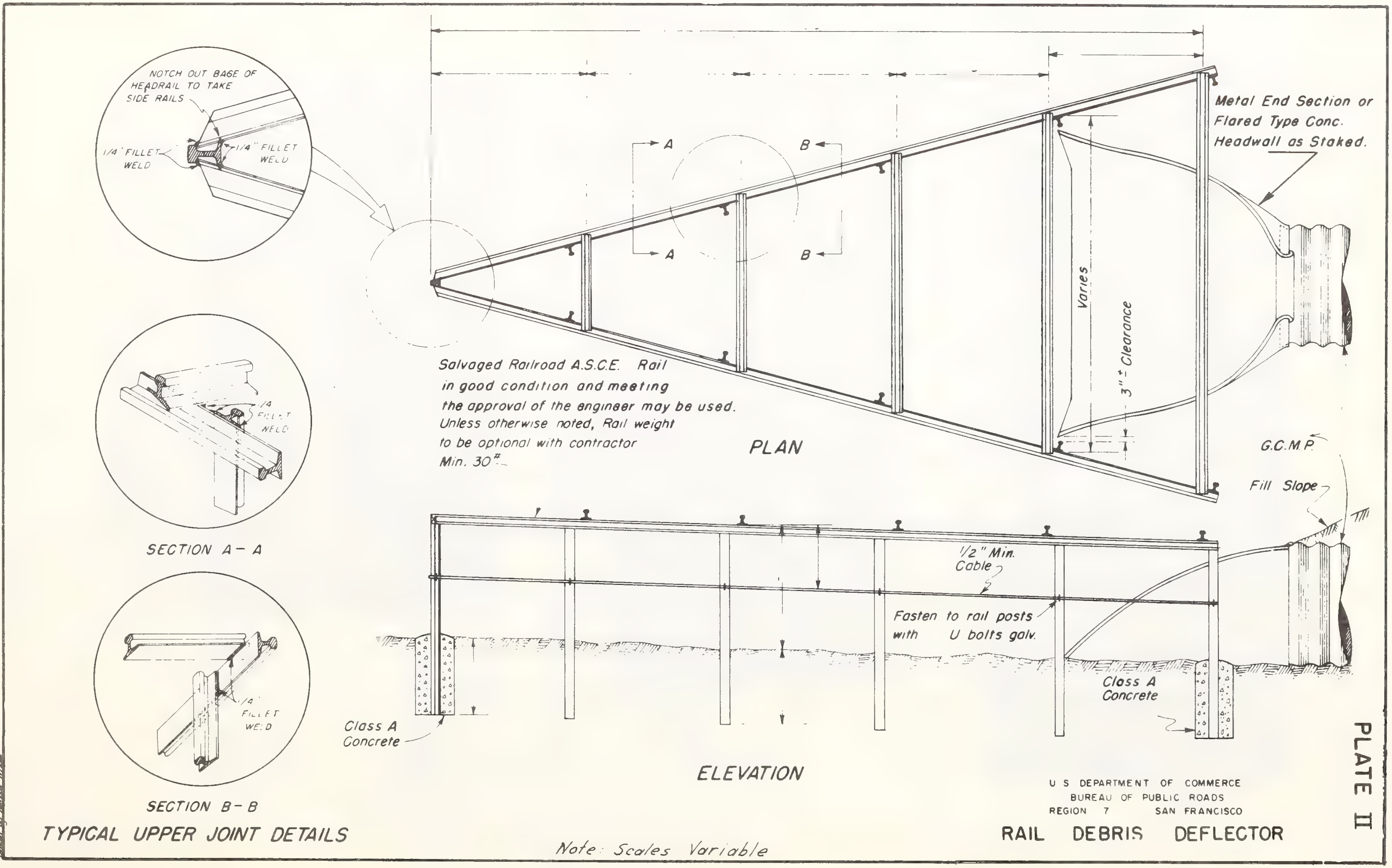
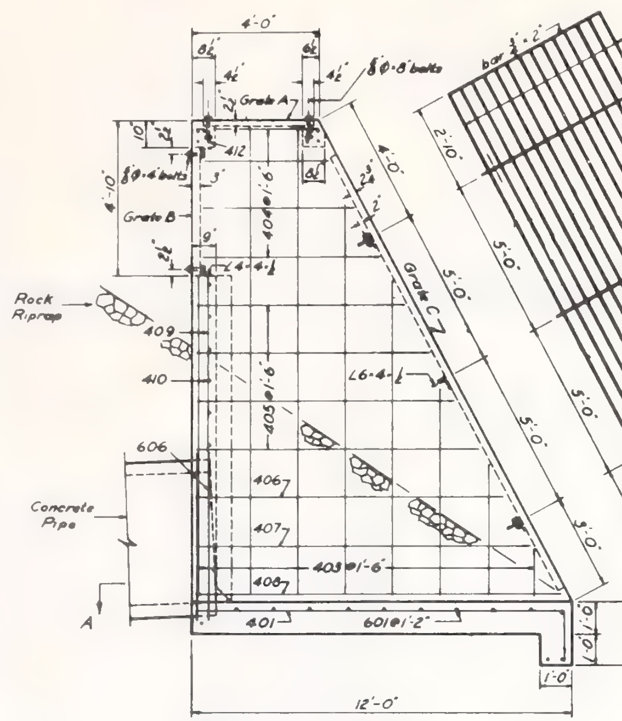


PLATE II
RAIL DEBRIS DEFLECTOR
U S DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
REGION 7 SAN FRANCISCO
PLAN
Metal End Section or
Flared Type Conc.
Headwall as Staked.
Varies
3"± Clearance
G.C.M.P.
Fill Slope
Salvaged Railroad A.S.C.E. Rail
in good condition and meeting
the approval of the engineer may be used.
Unless otherwise noted, Rail weight
to be optional with contractor
Min. 30#
ELEVATION
1/2" Min.
Cable
Fasten to rail posts
with U bolts galv.
Class A
Concrete
Class A
Concrete
NOTCH OUT BASE OF
HEADRAIL TO TAKE
SIDE RAILS
1/4" FILLET
WELD
1/4" FILLET
WELD
SECTION A-A
SECTION B-B
TYPICAL UPPER JOINT DETAILS
Note: Scales Variable

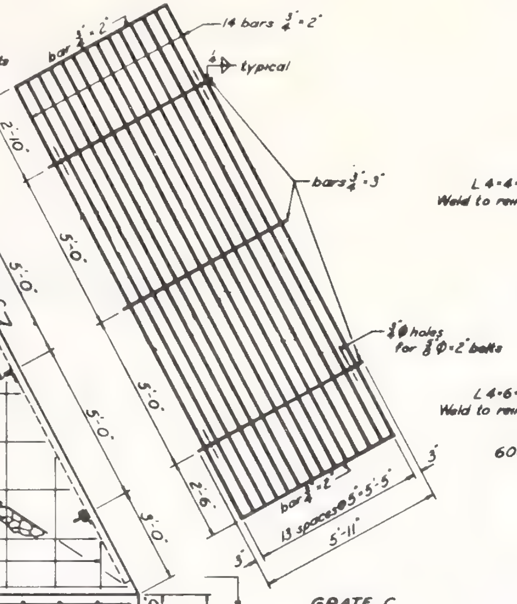
PLATE III

GENERAL NOTES
Concrete shall be Class A ($f'_c = 3600$)
Reinforcing steel shall be deformed bars of intermediate grade conforming to ASTM Specification A15
Structural steel shall be A-36

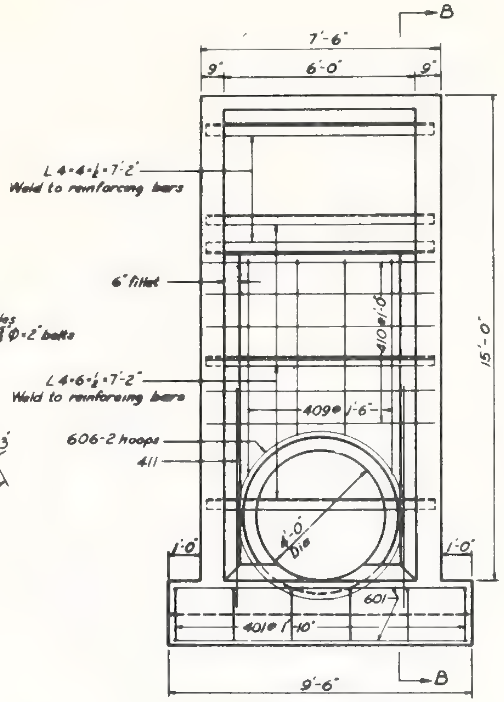
QUANTITIES	
Concrete	13 cu yds.
Reinforcing Steel	1540 lbs.
Structural Steel	2440 lbs.



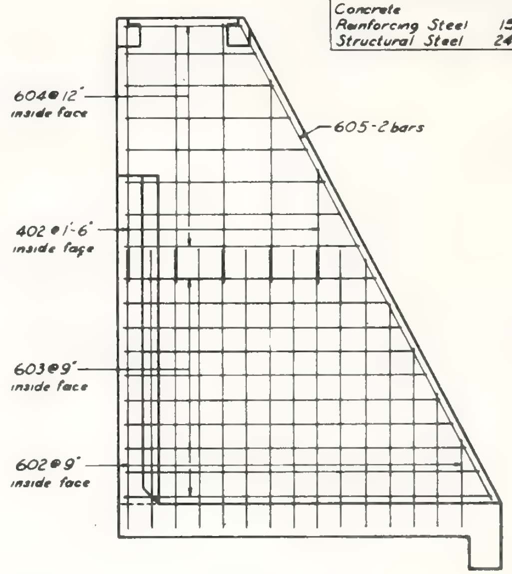
SIDE VIEW
outside face reinforcing shown



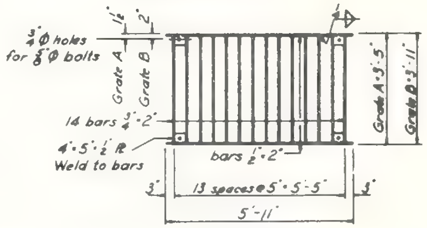
GRATE C



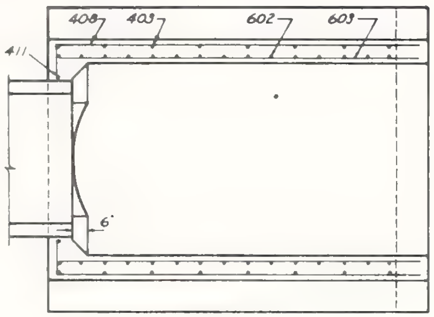
FRONT VIEW
grates not shown



SECTION B-B



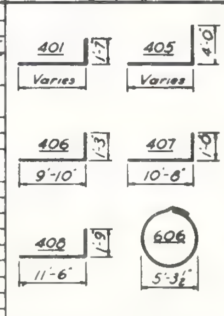
GRATES A & B



SECTION A-A

REINFORCING STEEL SCHEDULE						
Mark	Location	No.	Type	Size	Length	
601	Base	13	str	#6	9'-2"	
401	"	6	bent	4	Varies	2@12'-4", 4@13'-1"
602	Wall, inside face, vert.	30	str.	6	Varies	20@8'-9", 2 ea @8'-0", 6'-9", 5'-6", 4'-0", 2'-6"
'03	" " " horiz	20	str.	6	Varies	2 ea @11'-6", 11'-1", 10'-8", 10'-3", 9'-10", 9'-5", 9'-0", 8'-7", 8'-2", 7'-9"
604	" " " horiz	16	str.	6	Varies	2 ea @ 3'-10", 4'-4", 4'-10", 5'-4", 5'-10", 6'-4", 6'-10", 7'-4"
402	" " " vert	10	str	4	Varies	6@8'-0", 2@6'-0", 2@3'-6"
605	Wall, front slope	4	str	6	16'-8"	
403	Wall, outside face, vert	16	str	4	Varies	6@14'-8", 2 ea @13'-2", 10'-2", 7'-9", 4'-6", 2'-0"
404	" " " horiz	8	str	4	Varies	2 ea @ 6'-0", 5'-2", 4'-5", 3'-8"
405	" " " horiz	8	bent	4	Varies	2 ea @10'-9", 11'-6", 12'-3", 13'-0"
406	" " " horiz	2	bent	4	11'-1"	
407	" " " horiz	2	bent	4	11'-8"	
408	" " " horiz	2	bent	4	13'-3"	
606	Backwall, around pipe	2	bent	6	17'-7"	
409	" inside face, vert.	4	str	4	Varies	2@5'-6", 2@6'-9"
410	" " " horiz	6	str	4	7'-2"	
411	" outside face, vert	2	str	4	7'-0"	
412	Strut	8	str	4	7'-2"	

Bending Diagram



All dimensions are out to out

SPECIAL INLET – DEBRIS RACK

WASHINGTON STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
OLYMPIA, WASHINGTON

0 1 2 3
FEET

PLATE IV

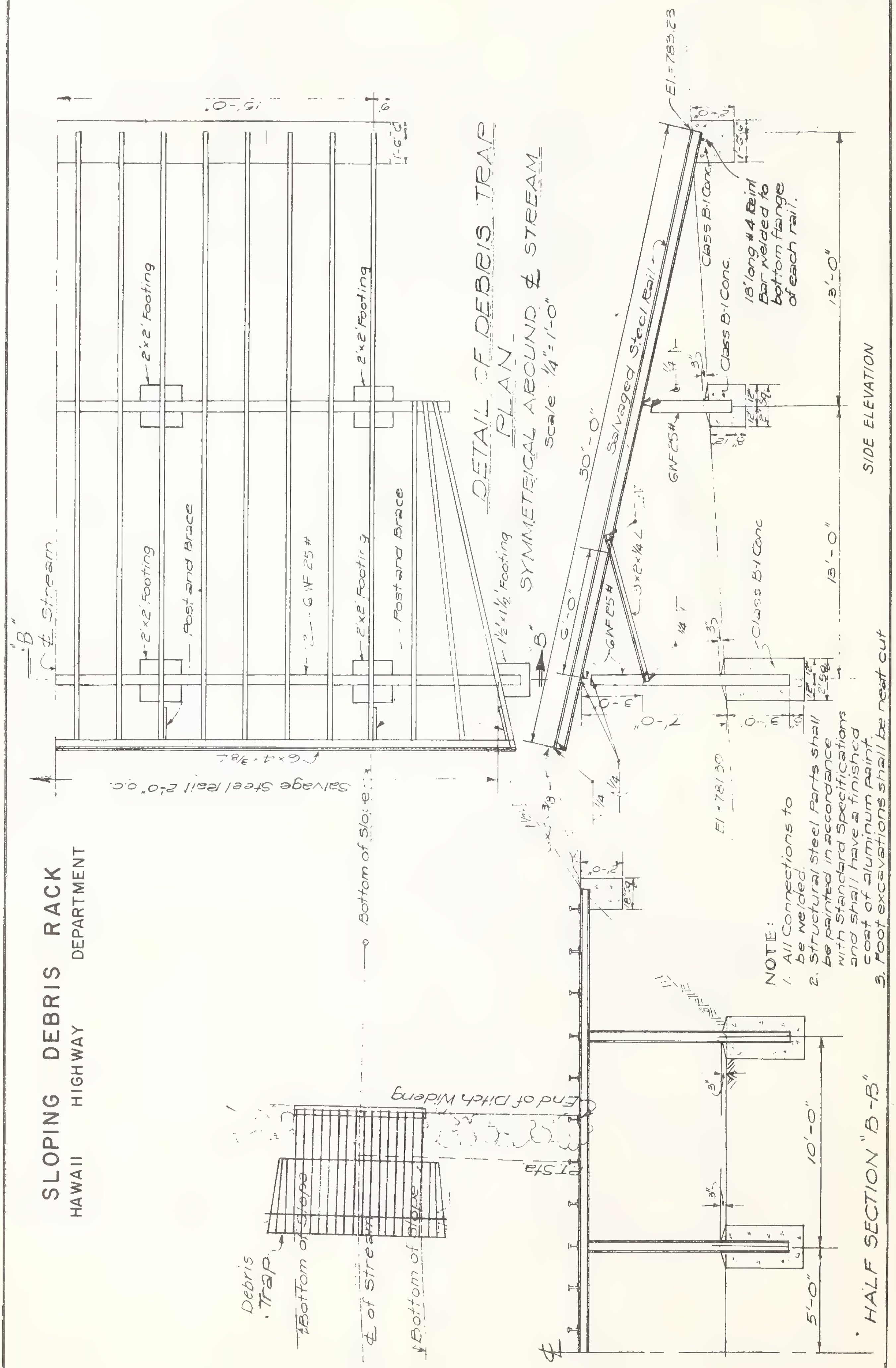
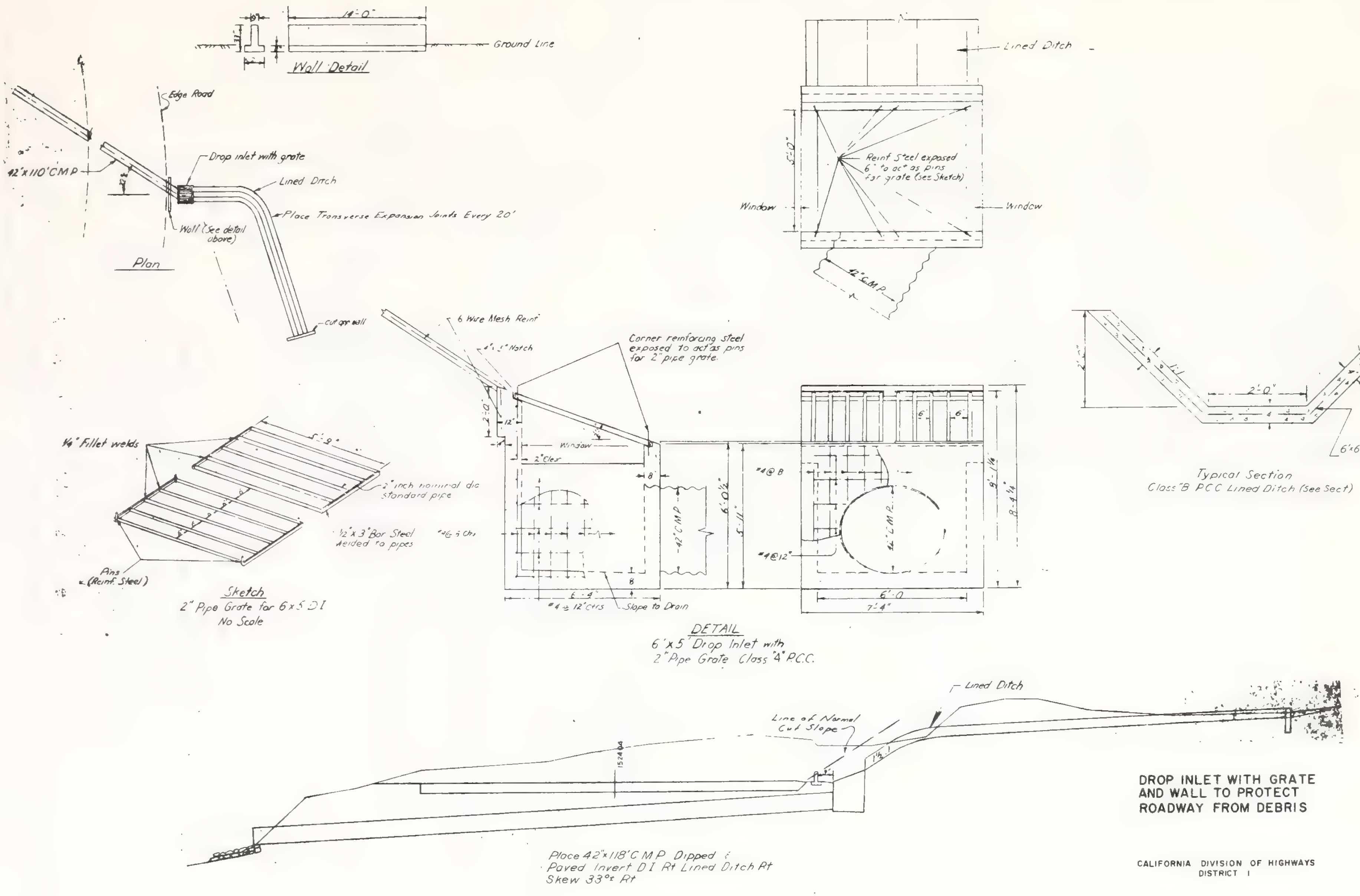

SLOPING DEBRIS RACK
HAWAII HIGHWAY DEPARTMENT
DETAIL OF DEBRIS TRAP
PLAN
SYMMETRICAL AROUND ℄ STREAM
Scale 1/4" = 1'-0"
SIDE ELEVATION
HALF SECTION "B-B"
Salvaged Steel Rail
Salvage Steel Rail 2'-0" o.c.
2'x2' Footing
Post and Brace
Class B-1 Conc.
18' long #4 Reinf Bar welded to bottom flange of each rail.
El = 783.23
El = 781.30
Bottom of Slope
℄ of Stream
Debris Trap
End of Ditch Widening
NOTE:
1. All Connections to be welded.
2. Structural Steel Parts shall be painted in accordance with Standard Specifications and shall have a finished coat of aluminum paint.
3. Foot excavations shall be neat cut

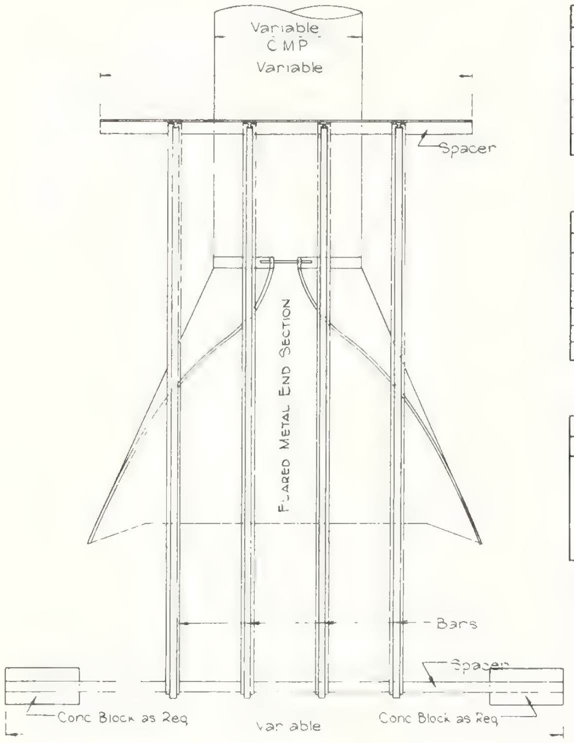
PLATE V



DROP INLET WITH GRATE
AND WALL TO PROTECT
ROADWAY FROM DEBRIS

CALIFORNIA DIVISION OF HIGHWAYS
DISTRICT I

PLATE VI



PLAN

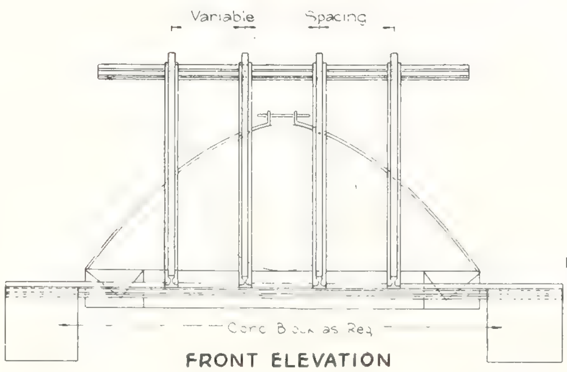
BAR SPACING FOR VARIOUS CULVERTS		
C.M.P.	BARS REQUIRED	BAR SPACING
18"	3	1'-0"
24"	4	1'-2"
30"	4	1'-4"
36"	4	1'-8"
42"	5	1'-8"
48"	5	1'-9"

REQUIRED LENGTH OF BARS			
C.M.P.	SLOPE OF BAR	BAR LENGTH	MATERIAL
18"	3 1	8'-3"	Std 3" Pipe or 25 to 40 Lb Rail
24"	3 1	11'-0"	" " " " "
30"	3 1	12'-0"	" " " " "
36"	3 1	13'-6"	40# to 60# Rail or Steel I's
42"	3 1	15'-0	" " " " " " "
48"	3 1	16'-6"	" " " " " " "

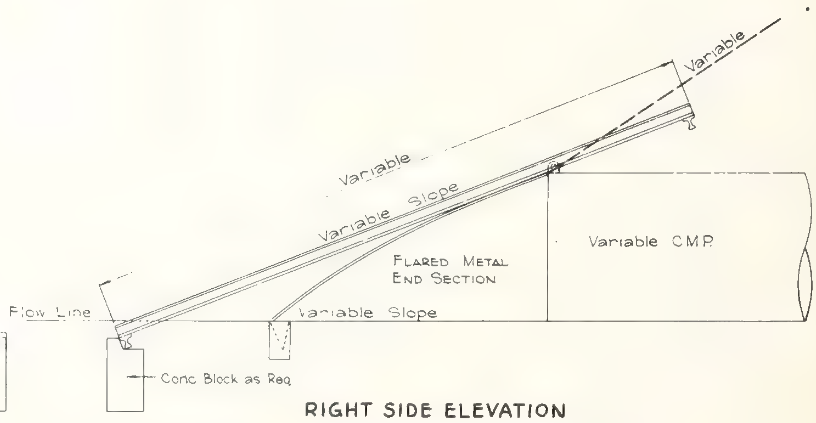
LENGTH OF SPACERS			
C.M.P.	TOP SPACER	BOTTOM SPACER	MATERIAL
18"	6'-0"	8'-0"	4"x4"x3/8" ∠s 40 to 60 Lb Rail or 3" Pipe
24"	7'-0"	10'-0"	
30"	7'-6"	11'-0"	
36"	8'-0"	12'-6"	
42"	9'-0"	13'-9"	
48"	10'-0"	15'-0"	

NOTE

- Special Treatment Required For Pipes Larger Than 48"
- Minimum Bar Spacing 0'-10"
- Maximum Bar Spacing 2'-0"
- Gradients Steeper Than 15% May Require Special Treatment
- Sizes Shown Are Minimums To Be Used.
- Heavier Sections Permitted In All Cases



FRONT ELEVATION

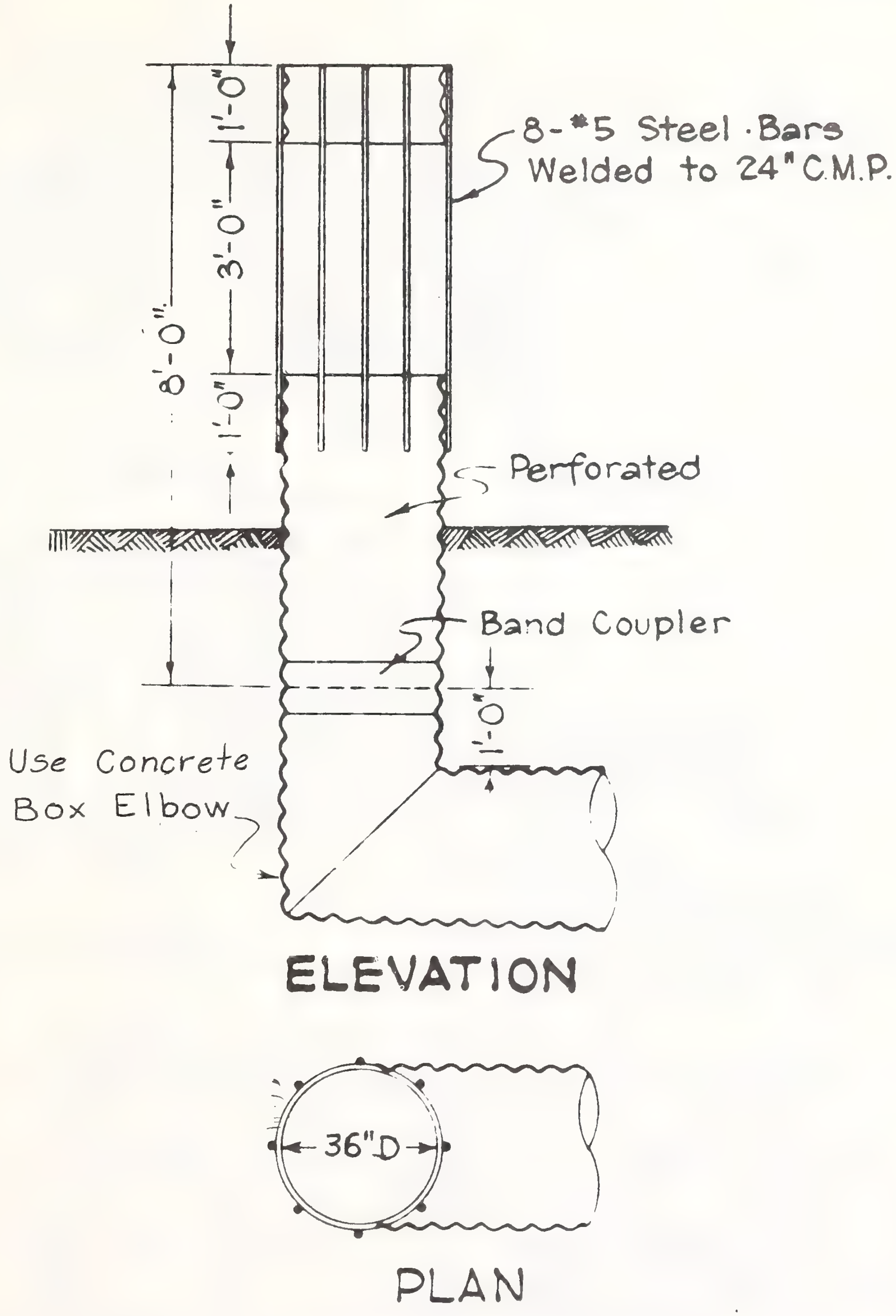


RIGHT SIDE ELEVATION

STANDARD DEBRIS RACK FOR 18" TO 48" C.M.P.'s

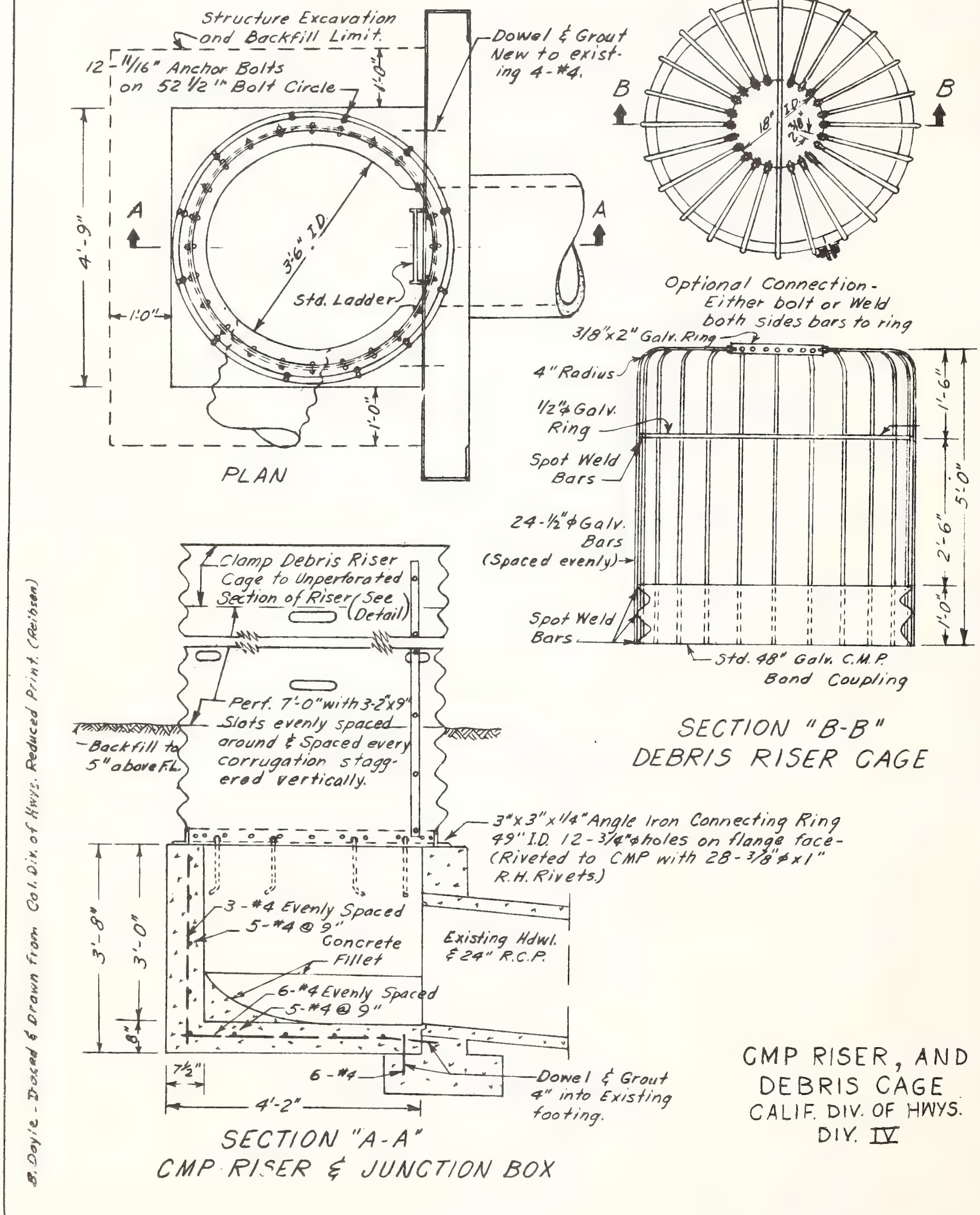
FOR USE WITH METAL END SECTION

CALIFORNIA DIVISION OF HIGHWAYS



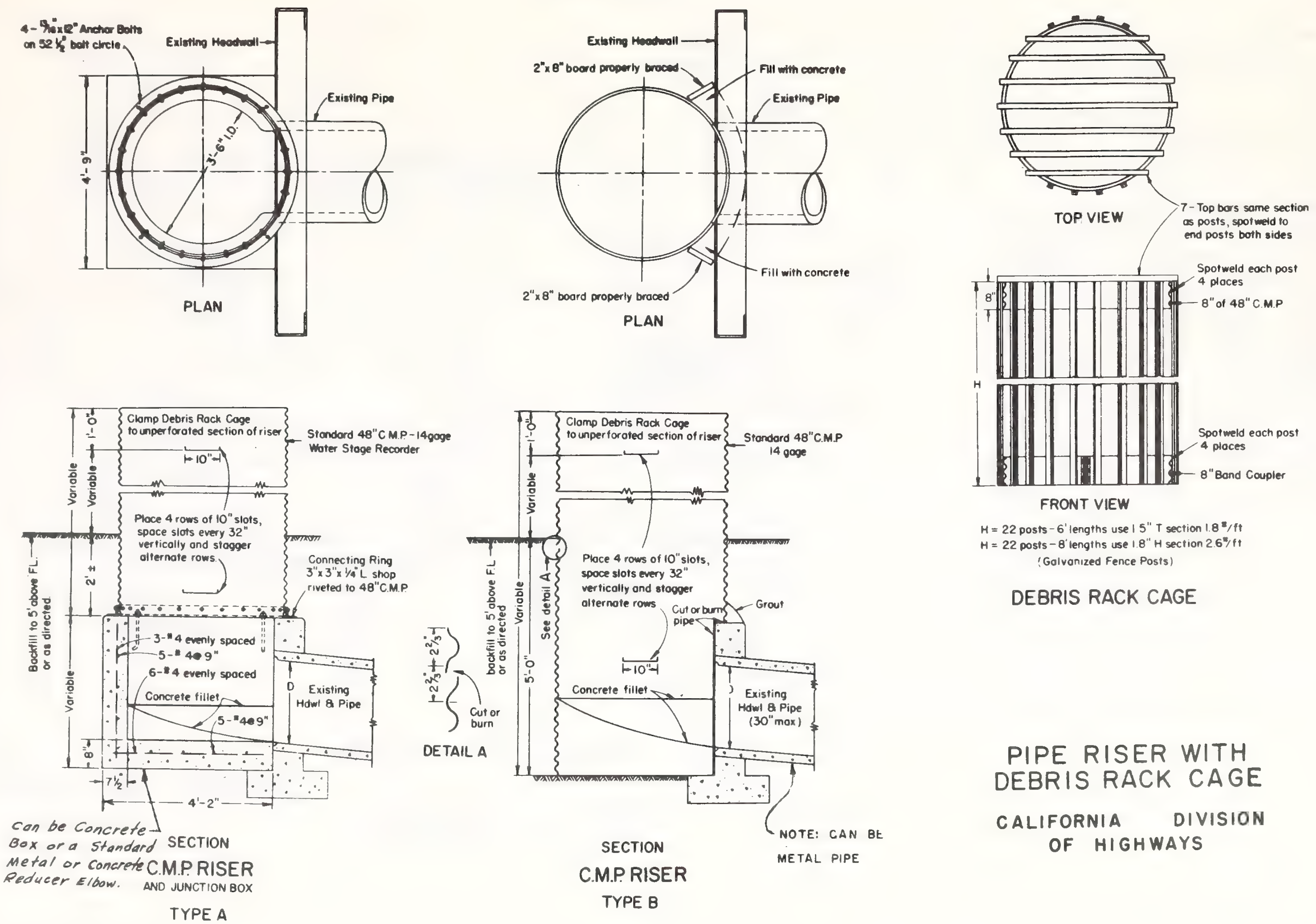
DEBRIS RISER

California Division of Highways
District II



CMP RISER, AND DEBRIS CAGE
CALIF. DIV. OF HWYS.
DIV. IV

PLATE IX

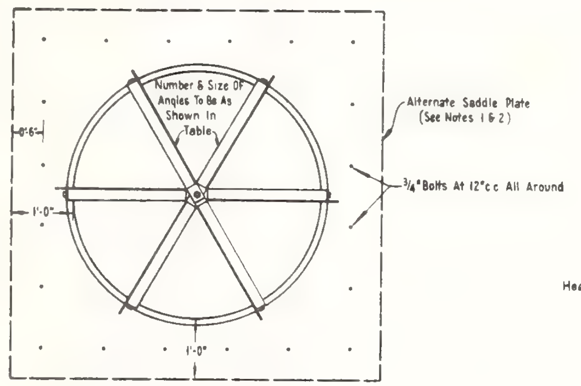


PIPE RISER WITH DEBRIS RACK CAGE

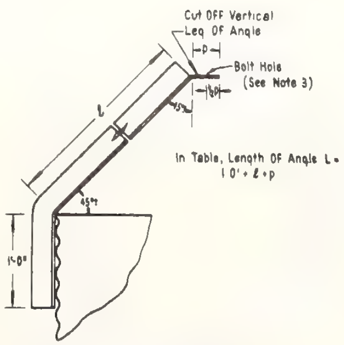
CALIFORNIA DIVISION OF HIGHWAYS

NOTES:

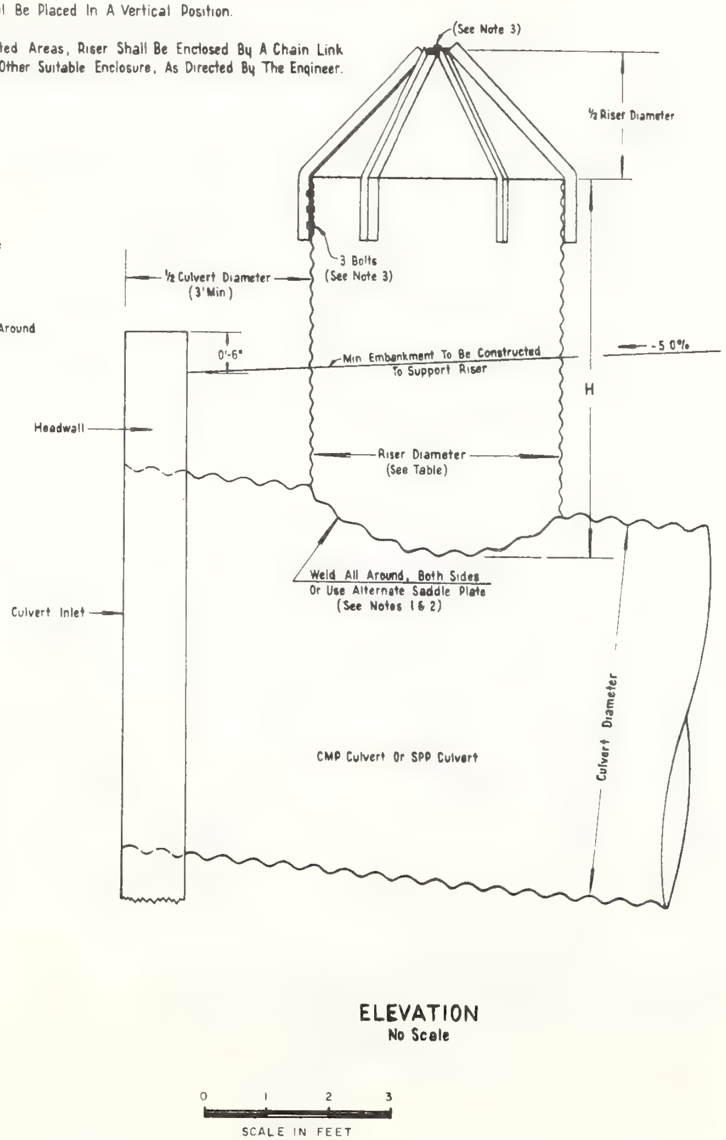
1. Riser To Be Fastened To Culvert By Welding Or Shop-Made Corrugated Metal Saddle Plate.
2. Saddle Plate Shall Be Same Gage Metal As Top Culvert Plates.
3. Cage Angles Shall Be Either Welded Or Bolted To Riser, And Top Joint Of Cage Angles Shall Be Either Welded Or Bolted. Bolt Diameter Shall Be Twice Angle Thickness.
4. All Angles, Nuts And Bolts Shall Be Galvanized
5. All Welds On Galvanized Metal Shall Be Treated With Zinc Dust In Accordance With Section 66-1.02 G Of The Standard Specifications.
6. Riser Shall Be Placed In A Vertical Position.
7. In Populated Areas, Riser Shall Be Enclosed By A Chain Link Fence Or Other Suitable Enclosure, As Directed By The Engineer.



TOP VIEW
No Scale



CAGE ANGLE DETAIL
No Scale



ELEVATION
No Scale

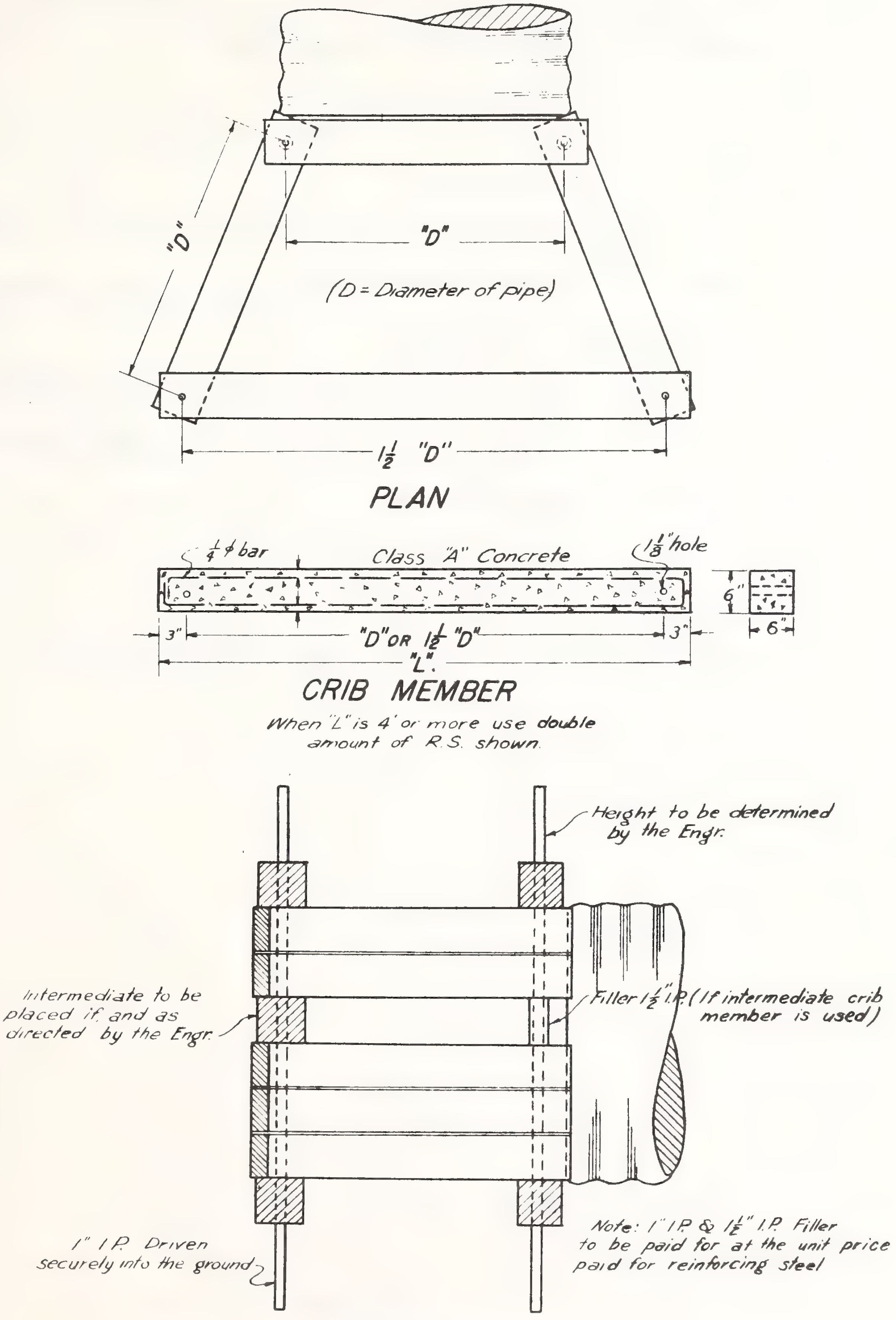
CULVERT DIAM.	RISER			RISER CAGE				
	C.M.P.		H*	STEEL				
	DIAM	GAGE		ANGLE SIZE	NO OF PIECES	LENGTH, FT.		
INCHES	INCHES		FEET			ℓ FT IN	p IN	L
36	24	14	4	2" x 2" x 1/4"	4	1'-5"	2"	2'-7"
42	24	14	4	2" x 2" x 1/4"		1'-5"	2"	2'-7"
48	30	14	4	2 1/2" x 2 1/2" x 1/4"		1'-10"	3"	3'-1"
54	36	12	4			2'-1"	3"	3'-4"
60	42	12	6			2'-6"	3"	3'-9"
66	42	12	6			2'-6"	3"	3'-9"
72	48	12	6			2'-10"	3"	4'-1"
78	48	12	6			2'-10"	3"	4'-1"
84	54	12	6	3" x 3" x 1/4"		3'-3"	3"	4'-6"
90	60	10	8			3'-6"	3"	4'-9"
96	60	10	8			3'-6"	3"	4'-9"
102	66	10	8			3'-11"	3"	5'-2"
108	72	10	8	3 1/2" x 3 1/2" x 5/16"		4'-3"	4"	5'-7"
114	72	10	8			4'-3"	4"	5'-7"
120	78	8	8			4'-8"	4"	6'-0"
126	84	8	10			5'-0"	4"	6'-4"
132	84	8	10			5'-0"	4"	6'-4"
138	90	8	10		6	5'-4"	7"	6'-11"
144	96	8	10			5'-8"	7"	7'-3"
150	96	8	10			5'-8"	7"	7'-3"
156	102†	12	12	4" x 4" x 3/8"	8	6'-0"	11"	7'-11"
162	108†	12	12			6'-5"	11"	8'-4"
168	108†	12	12			6'-5"	11"	8'-4"
174	114†	10	12			6'-9"	11"	8'-8"
180	120†	10	12			7'-1"	11"	9'-0"

* H May Be Varied. See Culvert Detail Sheets For Variable H.
† Structural Plate Pipe.

STANDARD RELIEF RISER
CALIFORNIA DIVISION OF HIGHWAYS

PLATE X

PLATE XI



ELEVATION

DEBRIS CRIB

CALIFORNIA DIVISION OF HIGHWAYS
DISTRICT 8

4.16 CULVERT DESIGN COMPUTER PROGRAMS

The following programs will aid in the design of various types of culverts. These descriptions explain the use of each program.

Name: SYP Language: Basic Input Format: Run

Purpose: Computes Head Losses Through Siphons

Rquired Input Data: Upstream Canal Velocity, Downstream Canal Velocity, Flow, Manning's "n", Total Siphon Length, Trial Siphon Diameter, and Trash Rack Dimensions.

Abstract: The program computes inlet and outlet losses, bend losses, friction losses and trash rack losses if trach racks are used. The individual and total head losses are printed out.

Limitations: Inlet and outlet structures must be similar to those shown on Form HYD-3.

The siphon must be similar to that shown in Figure 4.24 and on Form HYD-3.

Example Run

```
READY edit syp basic
EDIT  run
HEAD LOSSES THROUGH IRRIGATION SIPHONS

THE HEAD LOSSES COMPUTED HERE ARE BASED ON THE ASSUMPTIONS
THAT THE SIPHON AND TRANSITION STRUCTURES ARE SIMILAR TO THOSE
OF FORM HYD. 3 AND THAT THE TRASH RACKS ARE THOSE ON STD DWG 36

INPUT UPSTREAM CANAL VEL., DOWNSTREAM CANAL VEL., FLOW, MANNINGS N, SIPHON LENGTH
  ?   2.5,2.7,10,.012,140
INPUT BARREL DIA.(FT)
  ?   1.25
NO. OF TRASH RACKS?
  ?   0


FOR A 15 INCH RCP SIPHON BARREL

ENTRANCE LOSS =       0.28 FT
EXIT LOSS =           0.36 FT
BEND LOSSES =         0.12 FT
FRICTION LOSS =       2.86 FT
TRASH RACK LOSSES=    0.00 FT

TOTAL HEAD LOSSES =  3.62 FT


NEW DIA. ? 1=YES 2=NO
  ?   1
INPUT BARREL DIA.(FT)
  ?   1.5
NO. OF TRASH RACKS?
  ?   0


FOR A 18 INCH RCP SIPHON BARREL

ENTRANCE LOSS =       0.12 FT
EXIT LOSS =           0.15 FT
BEND LOSSES =         0.06 FT
FRICTION LOSS =       1.08 FT
TRASH RACK LOSSES=    0.00 FT

TOTAL HEAD LOSSES =  1.41 FT


NEW DIA. ? 1=YES 2=NO
  ?   1
INPUT BARREL DIA.(FT)
  ?   2
NO. OF TRASH RACKS?
  ?   1
INPUT BAR THICK., CTR TO CTR SPACING, APPROACH ANGLE
  ?   1,6,0


FOR A 24 INCH RCP SIPHON BARREL

ENTRANCE LOSS =       0.02 FT
EXIT LOSS =           0.02 FT
BEND LOSSES =         0.02 FT
FRICTION LOSS =       0.23 FT
TRASH RACK LOSSES=    0.02 FT

TOTAL HEAD LOSSES =  0.30 FT
```

4.17 STANDARD SIZES AND DIMENSIONS

The following tables list the culvert sizes that are commonly available and gives some of their basic data.

Table 4.13 Round Corrugated Steel Pipe

Diameter and Corrugations		End Area	Metal Thicknesses Available					
2 2/3" x 1/2"	3" x 1"	Sq. Ft.	.052	.064	.079	.109	.138	.168
12 in.		.79	X	X	X			
15		1.23	X	X	X			
18		1.77	X	X	X	X		
21		2.40	X	X	X	X		
23		3.14	X	X	X	X		
27		3.98		X	X	X		
30		4.91		X	X	X		
33		5.94		X	X	X		
36		7.1		X	X	X	X	
	36		X	X	X	X	X	X
42		9.6		X	X	X	X	X
	42		X	X	X	X	X	X
48		12.6		X	X	X	X	X
	48		X	X	X	X	X	X
54		16.0			X	X	X	X
	54		X	X	X	X	X	X
60		19.6			X	X	X	X
	60		X	X	X	X	X	X
66		23.8			X	X	X	X
	66		X	X	X	X	X	X
72		28.3				X	X	X
	72		X	X	X	X	X	X
78		33.2				X	X	X
	78		X	X	X	X	X	X
84		38.5					X	X
	84		X	X	X	X	X	X
90		44.2					X	X
	90		X	X	X	X	X	X
96		50.3					X	X
	96			X	X	X	X	X
	102	56.7		X	X	X	X	X
	108	63.6			X	X	X	X
	114	70.9			X	X	X	X
	120	78.5				X	X	X

Table 4.14 Corrugated Steel Arch Pipe

Size and Corrugation 2 2/3" x 1/2"	Size and Corrugation 3" x 1"	Equivalent Size	End Area Sq. Ft.	Metal Thicknesses Available .052	.064	.079	.109	.138	.168
17 x 13		15	1.1	X	X	X			
21 x 15		18	1.6	X	X	X	X		
24 x 18		21	2.2	X	X	X	X		
28 x 20		24	2.8	X	X	X	X		
35 x 24		30	4.4		X	X	X		
42 x 29		36	6.4		X	X	X	X	
	43 x 27	36	6.4	X	X	X	X	X	X
49 x 33		42	8.7		X	X	X	X	X
	50 x 31	42	8.7	X	X	X	X	X	X
57 x 38		48	11.4		X	X	X	X	X
	58 x 36	48	11.4	X	X	X	X	X	X
64 x 43		54	14.3			X	X	X	X
	65 x 40	54	14.3	X	X	X	X	X	X
71 x 47		60	17.6			X	X	X	X
	72 x 44	60	17.6	X	X	X	X	X	X
	73 x 55	66	22	X	X	X	X	X	X
77 x 52		66	21.3			X	X	X	X
	81 x 59	72	26	X	X	X	X	X	X
83 x 57		72	25.3				X	X	X
	87 x 63	78	31	X	X	X	X	X	X
	95 x 67	84	35	X	X	X	X	X	X
	103 x 71	90	40	X	X	X	X	X	X
	112 x 75	96	46		X	X	X	X	X
	117 x 79	102	52		X	X	X	X	X
	128 x 83	108	58			X	X	X	X
	137 x 87	114	64			X	X	X	X
	142 x 91	120	71				X	X	X

*"Pipe-arches made of pipe larger than 72 inches diameter may be of bolted contruction."

Table 4.15 Structural Steel Plate Pipe
(Available in thicknesses of .109, .138, .168, .188, .218, .249, .280)

Diameter Feet	End Area	Diameter Feet	End Area	Diameter Feet	End Area
5	19.6	10.5	86.6	16	201.1
5.5	23.8	11	95.0	16.5	213.8
6	28.3	11.5	103.9	17	227.0
6.5	33.2	12	113.1	17.5	240.5
7	38.5	12.5	122.7	18	254.5
7.5	44.2	13	132.7	18.5	268.8
8	50.3	13.5	143.1	19	283.5
8.5	56.7	14	153.9	19.5	298.6
9	63.6	14.5	165.1	20	314.2
9.5	70.9	15	176.7	20.5	330.1
10	78.5	15.5	188.7	21	346.4

Table 4.16 Structural Steel Plate Pipe-Arches
(Available in thicknesses of .109, .138, .168, .188, .218, .249, .280)

Span Ft.-In.	Rise Ft.-In.	Corner Radius	End Area	Span Ft.-In.	Rise Ft.-In.	Corner Radius	End Area
6-1	4-7	18	22	13-3	9-4	31	97
6-4	4-9	18	24	14-3	8-11	18	101
6-9	4-11	18	26	13-6	9-6	31	102
7-0	5-1	18	28	14-10	9-1	18	105
7-3	5-3	18	31	14-0	9-8	31	105
7-8	5-5	18	33	15-4	9-3	18	109
7-11	5-7	18	35	14-2	9-10	31	109
7-2	5-9	18	38	15-6	9-5	18	113
8-7	5-11	18	40	14-5	10-0	31	114
8-10	6-1	18	43	15-8	9-7	18	118
9-4	6-3	18	46	14-11	10-2	31	118
9-6	6-5	18	49	15-10	9-10	18	122
9-9	6-7	18	52	15-4	10-4	31	123
10-3	6-9	18	55	16-5	9-11	18	126
10-8	6-11	18	58	15-7	10-6	31	127
10-11	7-1	18	61	16-7	10-1	18	131
11-5	7-3	18	64	15-10	10-8	31	132
11-7	7-5	18	67	16-3	10-10	31	137
11-10	7-7	18	71	16-6	11-0	31	142
12-4	7-9	18	74	17-0	11-2	31	146
12-6	7-11	18	48	17-2	11-4	31	151
12-8	8-1	18	81	17-5	11-6	31	157
12-10	8-4	18	85	17-11	11-8	31	161
13-5	8-5	18	89	18-1	11-10	31	167
13-11	8-7	18	93	18-7	12-0	31	172
14-1	8-9	18	97	18-9	12-2	31	177
19-3	12-4	31	182	19-11	12-10	31	200
19-6	12-6	31	188	20-5	13-0	31	205
19-8	12-8	31	194	20-7	13-2	31	211

Table 4.17 Open Bottom Arches
(Available in thicknesses of .109, .138, .168, .188, .218, .249, .230 inch)

Span Feet	Rise Ft.-In.	Waterway Area In Sq. Ft.	Rise Over Span	Radius Inches
6.0	1-9 1/2	7 1/2	.30	41
	2-3 1/2	10	.38	37 1/2
	3-2	15	.53	36
7.0	2-4	12	.34	45
	2-10	15	.40	43
	3-8	20	.52	42
8.0	2-11	17	.37	51
	3-4	20	.42	48 1/2
	4-2	26	.52	48
9.0	2-11	18 1/2	.32	59
	3-10 1/2	26 1/2	.43	55
	4-8 1/2	33	.52	54
10.0	3-5 1/2	25	.35	64
	4-5	34	.44	60 1/2
	5-3	41	.52	60
11.0	3-6	27 1/2	.32	73
	4-5 1/2	37	.41	67 1/2
	5-9	50	.52	66
12.0	4-0 1/2	35	.34	77 1/2
	5-0	45	.42	73
	6-3	59	.52	72
13.0	4-1	38	.32	86 1/2
	5-1	49	.39	80 1/2
	6-9	70	.52	78
14.0	4-7 1/2	47	.33	91
	5-7	58	.40	86
	7-3	80	.52	84
15.0	4-7 1/2	50	.31	101
	5-8	62	.38	93
	6-7	75	.44	91
	7-9	92	.52	90
16.0	5-2	60	.32	105
	7-1	86	.45	97
	8-3	105	.52	96
17.0	5-2 1/2	63	.31	115
	7-2	92	.42	103
	8-10	119	.52	102
18.0	5-9	75	.32	119
	7-8	104	.43	109
	8-11	126	.50	108
19.0	6-4	87	.33	123
	8-2	118	.43	115
	9-5 1/2	140	.50	114
20.0	6-4	91	.32	133
	8-3 1/2	124	.42	122
	10-0	157	.50	120
21.0	6-11	104	.33	137
	8-10	140	.42	128
	10-6	172	.50	126
22.0	6-11	109	.31	146
	8-11	146	.40	135
	11-0	190	.50	132

Table 4.17 (Cont'd)

Span Feet	Rise Ft.-In.	Waterway Area In Sq. Ft.	Rise Over Span	Radius Inches
23.0	8-0	134	.35	147
	9-10	171	.43	140
	11-6	208	.50	138
24.0	8-6	150	.35	152
	10-4	188	.43	146
	12-0	226	.50	144
25.0	8-6 1/2	155	.34	160
	10-10 1/2	207	.43	152
	12-6	247	.50	150

Note: There are additional arches available with spans up to approximately 40 feet. However, their dimensions vary from manufacturer to manufacturer. The manufacturer's literature will have to be reviewed to determine the available sizes.

Table 4.18 Round Reinforced Concrete Pipe

Diameter In Inches	End Area In Sq. Ft.	Available Lengths In Feet	Wall Thickness In Inches	Weight Per Foot In Pounds	Available Classes
12	.79	4,6,8	2	92	II,III,IV,V
15	1.23	"	2 1/4	127	"
18	1.77	"	2 1/2	168	"
21	2.40	"	2 3/4	214	"
24	3.14	"	3	265	"
27	4.98	"	3 1/4	322	"
30	4.91	"	3 1/2	384	"
33	5.94	"	3 3/4	451	"
36	7.07	"	4	524	"
42	9.62	"	4 1/2	685	"
48	12.57	"	5	867	"
54	15.90	"	5 1/2	1071	"
60	19.63	"	6	1296	I,II,III,IV,V
66	23.76	"	6 1/2	1542	"
72	28.27	"	7	1810	"
78	33.18	"	7 1/2	2098	"
84	38.48	"	8	2408	"
90	44.18	"	8 1/2	2793	"
96	50.27	"	9	3092	"
102	56.75	4,6	9 1/2	3466	"
108	63.62	"	10	3864	"

Table 4.19 Reinforced Concrete Arch Pipe

Rise Inches	Span Inches	Equivalent Size Inches	End Area In Sq. Ft.	Available Lengths In Feet	Wall Thickness In Inches	Weight Per Foot In Pounds	Available Classes
13 1/2	22	18	1.65	4,6	2 1/2	170	II,III,IV
18	28 1/2	24	2.8	"	3 1/2	315	"
22 1/2	36 1/4	30	4.4	6,8	4	445	"
26 5/8	43 3/4	36	6.4	"	4 1/2	597	"
31 5/16	51 1/8	42	8.8	"	4 1/2	739	"
36	58 1/2	48	11.4	"	5	882	"
49	65	54	14.3	"	5 1/2	1090	"
45	73	60	17.7	4,6	6	1320	"
54	88	72	25.6	"	7	1840	"

Table 4.20 Reinforced Concrete Elliptical Pipe

Rise Inches	Span Inches	Equivalent Size Inches	End Area In Sq. Ft.	Available Lengths In Feet	Wall Thickness In Inches	Weight Per Foot In Pounds	Available Classes
48	76	60	20.5	6,8	6 1/2	1475	I,II,III,IV
53	83	66	24.8	"	7	1745	"
58	91	72	29.5	"	7 1/2	2040	"
63	98	78	34.6	"	8	2350	"
68	106	84	40.1	6	8 1/2	2680	"
72*	113	90	46.1	"	9	3050	"
77	121	96	52.4	"	9 1/2	3420	"
82*	128	102	59.2	"	9 3/4	3725	"
87	136	108	66.4	"	10	4050	"
92*	143	114	74.0	"	10 1/2	4470	"
97	150	120	82.0	"	11	4930	"

*Only these sizes are readily available in Montana.

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Saltzman, W. and R.O. Koski, Fish Passage Through Culverts Special Report, Oregon State Game Commission, July, 1971.

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Concrete Pipe Design Manual, First Edition, Prepared by American Concrete Pipe Association, Arlington, Virginia, 1970.

Certex Concrete Products, The Cretex Companies, Inc., Elk River, Minnesota, 1971.

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Hydraulic Engineering Circular No. 5, Hydraulic Charts for the Selection of Highway Culverts, U.S. Department of Commerce, Bureau of Public Roads, December, 1965.

Hydraulic Engineering Circular No. 9, Debris Control Structures, U.S. Department of Transportation, Bureau of Public Roads, December, 1969.

Hydraulic Engineering Circular No. 10, Capacity Charts for the Hydraulic Design of Highway Culverts, U.S. Department of Commerce, Bureau of Public Roads, March, 1965.

Hydraulic Engineering Circular No. 13, Hydraulic Design of Improved Inlets for Culverts, U.S. Department of Transportation, Federal Highway Administration, August, 1972.

SECTION 4.2
BRIDGE CROSSINGS

4.2 BRIDGE CROSSINGS

4.21 STREAMS AND RIVERS

Introduction

A hydraulic design is necessary for every crossing of a stream or river to determine the most economical bridge that will handle the design flow with a minimum of damage. Since most bridges constrict the natural channel, the primary design consideration is the resulting backwater and the extent of possible damage to the structure and upstream property as a result of such backwater.

Since the analysis for a bridge crossing involves a trial and error solution and several bridge lengths must be checked for each crossing, a computer analysis is warranted. Two methods of analyzing bridge crossings by computer are included in the Hydraulics Unit's program library. Discussions of the theories and methods included in these programs are beyond the scope of this manual. However, discussions of the applications and uses of each program are presented in Section 4.23.

Computer Programs

The two computer programs available for bridge analysis will both perform the required hydraulic computations, but they are based on different methods and they have different applications. A short discussion of the applications of each program is presented here and the actual use of the programs is presented in Section 4.23.

The first program is named HYM and is written in the PL1 language for terminal application. The program is based on the methods presented in Hydraulic Design Series No. 1, "Hydraulics of Bridge Waterways" published by the Federal Highway Administration in 1970. This is the easiest program to use and can be used for the majority of the bridge analyses. However, it does, have several limitations. The program does not have the capabilities to handle supercritical flow, multiple bridges, or backwater resulting from downstream conditions, such as highwater from

another stream or river. Whenever any of these conditions exist, the second program should be used.

The second program is named HEC-2 and is written in the fortran language for use with code sheets and punch cards. This program was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. HEC-2 uses Manning's equation along with the conservation of energy principle to compute the water surface profile between points or a series of points. The program has great versatility and with the use of its various options can be used to analyze almost any condition. However, the coding procedure is complex and this program should not be used unless conditions exclude the use of HYM.

Design Procedure

To insure that the bridge study is complete, the following basic steps should be followed:

Step 1. Collect and Analyze Site Data

Most of the site data should be provided on or with Form HYD-1. This information includes cross section, photographs, channel slope, possible flood damage, factors which could affect highwater stage, etc.

Step 2. Perform Hydrological Analysis

The design frequency, drainage area, and all other parameters that may be necessary in the analysis must be determined before the design discharge can be determined. The design frequency used depends upon the location of the crossing, the road system, and the magnitude of possible damage to adjoining property or the highway from floods of a greater magnitude. Using one of the methods outlined in Section 3, Hydrology, calculate the design flood and the basic flood for the crossing. In most cases, the design flood should be calculated by more than one method with the results analyzed to determine the method that best defines the area based on its flood history and the performance of existing structures during past events.

Step 3. Prepare Computer Input Data

All of the data required by the computer to do the hydraulic computations must be prepared for input. This data includes the cross section data, hydraulic data, constriction data, and bridge information. An exact description of the required data is given in Section 4.23.

Step 4. Make Hydraulic Computations on Computer

Step 5. Consider Erosion Control

If stream velocities through the bridge opening exceed 3 ft/sec then the need for rip rap and a filter blanket should be investigated. When approach embankments encroach on wide flood plains and constrict the normal flood flow, the need for spur dikes should also be investigated. The design procedure for rip rap, filter blankets, and spur dikes are presented in Section 4.8, Erosion Control.

Step 6. Analyze Output and Make Recommendations

Review the hydraulics data computed for the various bridge lengths for the design flood and make selection of required opening based on potential flood damage, no inundation of the roadway, economics, stream velocities, and clearances.

The selected opening shall then be evaluated for possible damage to adjoining landowners or the highway and delays to the public resulting from the basic flood. Also any encroachment or constriction of a delineated floodway must be evaluated to insure that the resulting flood water elevation during the basic flood is in compliance with the criteria of the Montana Floodway Management and Regulation Act.

Step 7. Send Recommendations to Bridge Bureau

The hydraulic recommendations shall be transmitted to the Bridge Bureau by inter-departmental memorandum, using the format of Figure 4.28. The terms

used in Figure 4.28 are illustrated on Figure 4.29.

Documentation

Form HYD-4, "Bridge Crossing Hydraulic Report", shall be filled out completely and accurately to define the site location, to show any design assumptions that have been made, and to show the methods used in the analysis. This form along with a copy of Form HYD-1, design computations, computer print-outs, and copies of all correspondence shall be placed in the project files.

4.22 IRRIGATION

The hydraulic design of a bridge crossing of an irrigation canal is normally quite simple. It is usually necessary to completely span the canal so, except for very large canals where a pier may be required, there is no constriction. The normal water surface elevation must be determined and the backwater caused by a pier, if one is required, must be computed. Computer program "HYM" will perform these calculations.

ORM 4 A

INTER-DEPARTMENTAL MEMORANDUM

DEPARTMENT OF HIGHWAYS

To Chief-Bridge Bureau

From Manager-Hydraulics Unit

Date

Subject

As requested by your memorandum dated January 3, 1975, we are submitting the following recommendations for bridge crossings:

Station 10+00		
Drainage Area	25 Square Miles	
Design Frequency	50 Year	
Design Discharge	2000 c.f.s.	
Centerline Channel	Station 9+20	
Channel Bottom Width	20 Feet	
Channel Bottom Elevation	2016.9	
Channel Slope	0.004 Ft./Ft.	
Stage Elevation	2020.5	
Basic Flood (100 yr.)	2400 c.f.s.	
Basic Flood Stage Elevation	2020.8	
	Design Flood	Basic Flood
Backwater (no piers)	0.2 Feet	0.3 Feet
Backwater (1 pier)	0.3 Feet	0.4 Feet
Backwater (2 piers)	0.4 Feet	0.5 Feet
Velocity	7.5 f.p.s.	7.7 f.p.s.
Skew	0 Degrees	
Bank Protection	Class II Rip Rap	
Embankment Slope	2:1	
Spur Dikes	None required	

To obtain the highwater elevations, the backwater depths for the number of piers used must be added to the Appropriate Stage Elevation.

We are attaching Form HYD-1 and the survey notes for your use and file.

Manager-Hydraulics Unit

34- : :sk

Attachment

cc: Supvr.-Location and Road Design Section
Chief-Preconstruction Bureau

Figure 4.28

Avoid Verbal Instructions

3

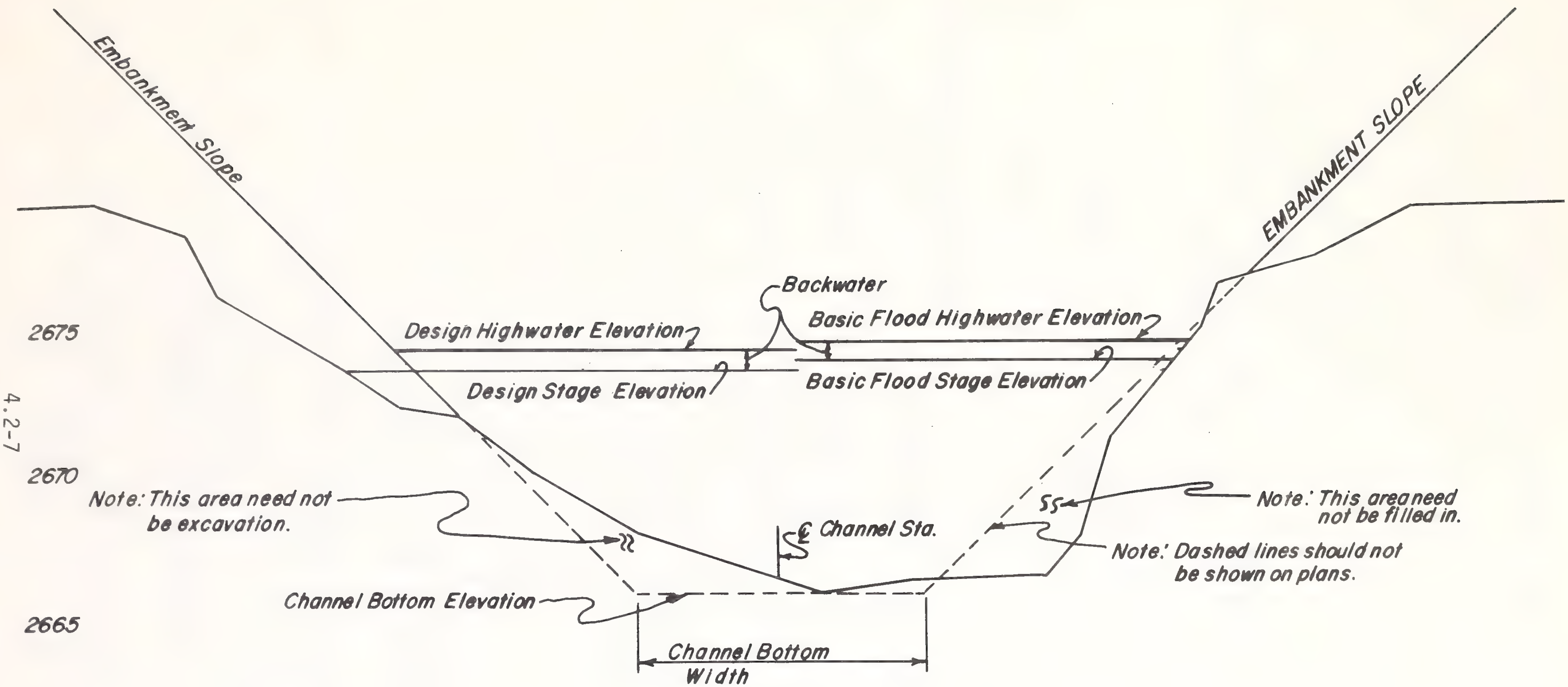

Embankment Slope
EMBANKMENT SLOPE
2675
2670
2665
Backwater
Design Highwater Elevation
Design Stage Elevation
Basic Flood Highwater Elevation
Basic Flood Stage Elevation
Note: This area need not be excavation.
℄ Channel Sta.
Note: This area need not be filled in.
Note: Dashed lines should not be shown on plans.
Channel Bottom Elevation
Channel Bottom Width
791+00
+20
+40
+60
+80
792+00
+20
+40
+60
+80
793+00
+20

FIGURE 4.29

Form HYD-4

BRIDGE CROSSING HYDRAULIC REPORT

Project Name________________________ No.____________

Stream Name________________________ Sta.____________

Designer________________________ Date____________

A. SITE DATA

1. Attach Location Map - Indicate crossing location
2. Attach A Copy Of Completed FORM HYD. 1
3. Attach Photographs
4. Discuss Factors Affecting Water Stages
   a. High water from other streams________________________

      ____

      ____

   b. Reservoirs & Lakes________________________

      ____

      ____

   c. Springs________________________

      ____

      ____

   d. Other________________________

      ____

      ____

B. HYDROLOGICAL ANALYSIS

1. Drainage Area__________ Sq. Mi.

   How was drainage area determined?________________________

   ____

   ____

   Describe drainage basin parameters________________________

   ____

   ____

2. Design Frequency________yrs. Design Flow_______cfs

By which method was the design flow determined?_______

____

____

3. 100-year Flow_________cfs. By which method was the 100-year flow determined?_____________________

____

____

4. Historic Highwater Elevation__________ Flow___________

Frequency___________ Date_____________ Cause__________

____

C. <u>HYDRAULIC ANALYSIS</u>

1. Permissible Backwater Elevation_______________

2. Channel Slope__________ft/ft How was slope determined?

____

3. Attach Stage-Discharge Data (Computer Output) For Natural Channel

4. Attach Backwater Computations (Computer Output) For Various Bridge Lengths For Design Flow

5. Final Recommendations

a. Station at left channel bottom_____________________

Station at right channel bottom____________________

b. Channel bottom width_________ Channel bottom elevation ____________

c. Fill slopes__________ Skew_____________

d. No. of piers_________ Type of piers______________

e. Normal stage elevation___________ Backwater_________

Calculated highwater elevation ____________________

f. Design velocity____________ft/sec

g. Bank protection

Is riprap required?____________ Type_______________

Is filter blanket required?__________ Describe_____

____

____

h. Channel changes___________________________________

____

____

i. Spur dikes____

____

____

6. Attach Backwater Computations (Computer Output) For 100-year Flow

7. Discuss The Effect Of The 100-year Flow On The Highway, Stream, And Other Property____________________________

____

____

____

____

____

____

____

____

4.23 BRIDGE DESIGN COMPUTER PROGRAMS

The following programs will perform the hydraulic analysis for bridge crossings:

Name: HYM Language: PL1 Input Format: Run

Purpose: Computes Stage-Discharge, Normal Stage, and Backwater for Bridges.

Required Input Data: Number of cross section points used to describe channel. horizontal distance, elevation, and Manning's "n" for each point, low stage, high stage, and increment for Stage-discharge computations, channel slope, design flow, left and right toe distance for each constriction, fill slope, skew angle in degrees, number of piers, pier type: ○-#1, ⊂⊃-#2, ○○-#3, ○○○○○-#4, □□□□□-#5

Abstract: This program computes the stage discharge information from the low stage to the high stage with the increment given. It then computes the backwater based on the constriction data given. The stage-discharge calculations are based on Manning's equation and the method for the backwater computation is taken from the Hydraulic Design Series No. 1 entitled "Hydraulics of Bridge Waterways" published by the Federal Highway Administration in 1970.

Limitations: The program will not work for super critical flow, it will not handle multiple structures, and it will not handle varied flow. A maximum of 25 points may be used to describe the channel cross section.

Example Run

```
      edit hym ipl1
EDIT  run
NO.OF POINTS
?     19
X,Y,ROUGHNESS COEF.
?     30,20,.05
?     70,20,.05
?     90,16,.04
?     120,14,.04
?     130,12,.035
?     140,10.8,.035
?     150,12,.04
?     160,14,.04
?     170,14.8,.04
?     180,14,.04
?     190,12,.035
?     200,8,.032
?     210,7.3,.032
?     220,8,.035
?     240,10,.04
?     260,12,.04
?     280,16,.05
?     290,20,.05
?     300,21,.05
LOW STAGE,HIGH STAGE,INCREMENT,SLOPE,DESIGN FLOW
?     10,20,1,.0035,9000

              **CHANNEL FLOW CHARACTERISTICS**
          STAGE          Q              AREA            VEL
          10.00        290.73           72.00          4.04
          11.00        614.57          123.58          4.97
          12.00       1092.74          199.00          5.49
          13.00       1819.49          299.00          6.09
          14.00       2782.30          419.00          6.64
          15.00       4042.74          571.00          7.08
          16.00       5642.17          751.00          7.51
          17.00       7622.42          944.75          8.07
          18.00       9923.83         1146.00          8.66
          19.00      12533.61         1354.75          9.25
          20.00      15443.91         1571.00          9.83
NEW STAGE DATA OR SLOPE?
?     nop
             ***NORMAL STAGE CHARACTERISTICS***
          17.63       9024.12         1069.65          8.44
```

(cont)

```
**CONSTRICTION DATA**
LT AND RT TOE DISTANCE,FILL SLOPE
?      170,280,2
FLOW LT OF OPENING                          =      2089.64 CFS
FLOW THRU OPENING                           =      6933.42 CFS
FLOW RT OF OPENING                          =         1.06 CFS
BRIDGE OPENING LENGTH AT WATER SURFACE      =       118.90 FT


SKEW ANGLE
?      10
PIERS?
?      yes
NOS OF PIERS
?      1
X-COORD OF PIER,PIER WIDTH,--,--
?      225,3
PIER TYPE
?      2


VELOCITY THRU CONSTRICTION                  =        12.62 FT/SEC
AREA OF CONSTRICTION                        =       715.00 SQ FT
AREA OF PIERS                               =        27.38 SQ FT
BACKWATER                                   =         2.13 FT
DIFFERENT PIERS?
?      yes
NOS OF PIERS
?      2
X-COORD OF PIER,PIER WIDTH,--,--
?      200,3,240,3
PIER TYPE
?      2


VELOCITY THRU CONSTRICTION                  =        13.07 FT/SEC
AREA OF CONSTRICTION                        =       690.62 SQ FT
AREA OF PIERS                               =        51.75 SQ FT
BACKWATER                                   =         2.38 FT
DIFFERENT PIERS?
?      nop
```

(con't)

```
NEW CONSTRICTION DATA?
?      yes


**CONSTRICTION DATA**
LT AND RT TOE DISTANCE,FILL SLOPE
?      120,280,2
FLOW LT OF OPENING                        =        299.89 CFS
FLOW THRU OPENING                         =       8723.17 CFS
FLOW RT OF OPENING                        =          1.06 CFS
BRIDGE OPENING LENGTH AT WATER SURFACE    =        170.50 FT


SKEW ANGLE
?      10
PIERS?
?      yes
NOS OF PIERS
?      1
X-COORD OF PIER,PIER WIDTH,--,--
?      190,3
PIER TYPE
?      2


VELOCITY THRU CONSTRICTION                =          9.21 FT/SEC
AREA OF CONSTRICTION                      =        979.91 SQ FT
AREA OF PIERS                             =         16.88 SQ FT
BACKWATER                                 =          0.13 FT
DIFFERENT PIERS?
?      yes
NOS OF PIERS
?      2
X-COORD OF PIER,PIER WIDTH,--,--
?      170,3,230,3
PIER TYPE
?      2


VELOCITY THRU CONSTRICTION                =          9.38 FT/SEC
AREA OF CONSTRICTION                      =        962.43 SQ FT
AREA OF PIERS                             =         34.35 SQ FT
BACKWATER                                 =          0.27 FT
DIFFERENT PIERS?
?      yes
NOS OF PIERS
?      3
X-COORD OF PIER,PIER WIDTH,--,--
?      160,3,200,3,240,3
PIER TYPE
?      2


VELOCITY THRU CONSTRICTION                =          9.66 FT/SEC
AREA OF CONSTRICTION                      =        934.16 SQ FT
AREA OF PIERS                             =         62.63 SQ FT
BACKWATER                                 =          0.49 FT
DIFFERENT PIERS?
?      nop
NEW CONSTRICTION DATA?
?      nop
NEW STAGE DATA OR SLOPE?
```

Name HEC-2 Language: Fortran Input Format: Code Sheets

Purpose: Computes Water Surface Profiles for River Channels

Required Input Data: The required input data and coding procedure are much to complex to detail here. The "Users Manual" should be used when coding a project.

Abstract: This program entitled "HEC-2 Water Surface Profiles" was written by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The program computes and plots the water surface profile for river channels of any cross section for either sub critical or super critical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments, and dams may be considered in the computations. River conditions such as variable channel roughness, islands, bends, levee overflow, river confluences, and water falls may also be included. The "Users Manual" must be consulted when using this program.

Limitations: See Users Manual

Example Run: See Users Manual

REFERENCES

Federal-Aid Highway Program Manual - Volume 6, Chapter 7, Section 3, Subsection 2 - Hydraulics Design of Highway Encroachments on Flood Plain.

HEC - 2 Water Surface Profiles, Users Manual, U.S. Army, Corps of Engineers, The Hydrologic Engineering Center, Davis, California, February, 1972.

Hydraulic Design Series No. 1, Hydraulics of Bridge Waterways, Second Edition, U.S. Department of Transportation, Federal Highway Administration, 1970.

Montana Floodway Management and Regulation Act, (Section 89-3501 through 89-3515, R.C.M., 1947).

SECTION 4.3
WATER SUPPLY

4.3 WATER SUPPLY

Introduction

Water supply systems are required to provide water for domestic use in rest areas and weigh stations and for sprinkler systems on rest areas and landscape projects. The primary source of water in most cases will be a well. Design of a complete water supply system includes the design of the pumping, pressure, and purification systems. Design of each of these systems plus descriptions of alternate component parts are discussed separately in the following paragraphs.

4.31 PUMPING SYSTEM

Design of the pumping system consists of determining the water demand; specifying the type and size of pump required to supply the water demand at design pressure; specifying type and size of pipes, valves and other related materials required to complete the system; and in some cases determining the total pumping head.

Water Demand (Rest Areas)

Only the water demand required by the comfort station facilities, i.e., drinking fountains, lavatories, etc. will be discussed in this section. Water demand for rest area sprinkler systems will be discussed in Section 4.4, "Sprinkler Systems". The comfort station water demand design procedures presented here are taken from Robert H. Baumgardner's paper entitled, "Rest Area Sewage Disposal and Water Supply". This paper was presented to the Roadside Development Session of the 1972 AASHO Meetings.

The design daily flow rate is given by the following formula:

$$Qd = ADT \times DF \times S \times P \times SPK \times G$$

Where Qd = Design daily flow rate gal/day

ADT = Estimated Average Daily Traffic for the design year

DF = Direction Factor

1.0 = if both directions served

0.5 = if only one direction served

S = Percent of vehicles stopping at the rest area per day

P = Number of people per vehicle using comfort facilities (usually 2.5)

SPK = Seasonal peak factor (usually 1.30)

G = Number of gallons per user (usually 5 gallons)

Studies have shown that 2.25 persons per car will use the comfort station facilities and that between 5 and 14 percent of the passing traffic will stop at the rest area, with the higher percentages occurring at the more remote locations. The purpose of the seasonal peak factor is to design the rest area for a peak

season rather than a peak hour or day. The average amount of water used by each user is estimated at five gallons.

The peak hourly flow rate is calculated by the following formula:

$$Qh = Qd \times k \times D$$

Where Qh = Peak hourly rate for flow gal/hr.

Qd = Design daily flow rate

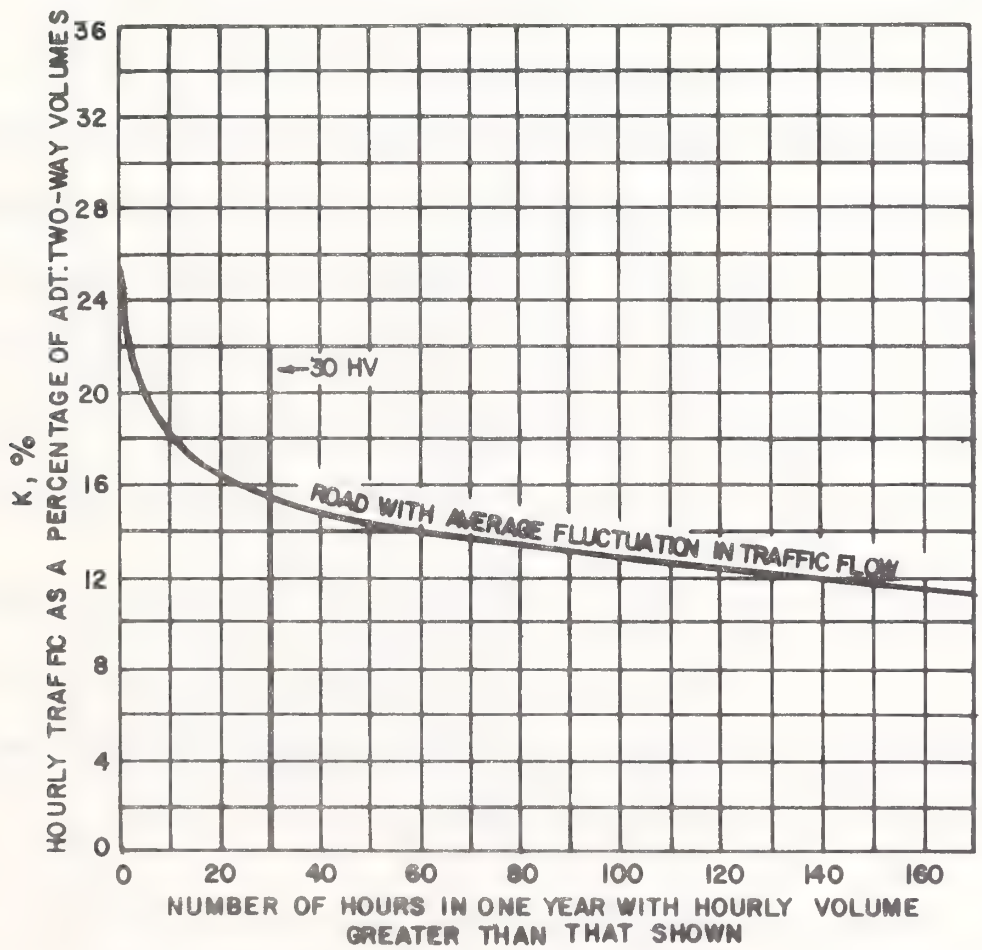
K = Factor representing portion of daily flow occurring during peak hour (usually .15)

D = Direction distribution allowance

1.0 = if both directions served

1.2 = if only one direction served

Figure 4.30, which is taken from "A Policy on Geometric Design of Rural Highways - 1965" by AASHO, will serve as a guide in selecting values of K.



RELATION BETWEEN PEAK HOUR AND AVERAGE DAILY TRAFFIC VOLUMES
MAIN RURAL HIGHWAYS

FIGURE 4.30

The peak hourly rate of flow should be checked against the well capacity to insure there is adequate water. If the well capacity is less than Q_h, consideration should be given to the use of a storage tank.

Water Demand (Weigh Station)

A typical weigh station requires water for one toilet, one lavatory, and one hose bib. A minimum water demand of 5 GPM may be used for design. However, when plenty of water is available, a water demand of 10 GPM should be used. These figures should be doubled when two weigh stations are operated from one well and pump.

Water Demand (Landscaping Projects)

Water demand will be that required for the sprinkler system as discussed in Section 4.4, "Sprinkler Systems".

Well Capacity

The maximum sustained output of the well must be determined so that this output is never exceeded. The maximum output can be determined from the test pumping log recorded on Form HYD-2. By plotting the drawdown versus the flow rate for different pumping rates, a curve can be plotted showing the probable drawdown for any pumping rate. Such a curve is valuable for determining what depth the pump should be set, as well as determining the well capacity.

Consideration should be given to the amount of fluctuation in water table elevations. If there is a great deal of fluctuation in the water table, the well capacity should be determined or at least estimated for the time of year that the water table is at its lowest point.

Types of Pumps

There are four types of pumps that could possibly be used for rest area water supply: submersibles; centrifugals, and shallow well jets; turbines; and deep well jets. A short discription of each type and a figure showing a typical

installation follow:

Submersibles

The submersible pump is by far the most common installation. Because both the pump and motor are placed in the well casing, there is no need for a pump-house and there is no danger of the pump freezing. The well also muffles the noise of the pump. The primary disadvantage of the submersible is it is difficult to pull the pump out of the well to work on it. Figure 4.31 shows a typical submersible pump installation.

Centrifugals and Shallow Well Jets

When the pumping depth to water does not exceed 20 feet, a centrifugal or shallow well jet may be used. The main advantage of this type of pump is its accessibility. However, this type of pump must be placed in the comfort station or a wellhouse or freezing and vandalism could be problems. It is also necessary to check the net positive suction head available to see that it is greater than the net positive suction head required. The NPSHA is computed by the following formula:

$$NPSHA = Ho - Hv - h - hsl$$

Where NPSHA = Net Positive Suction Head Available

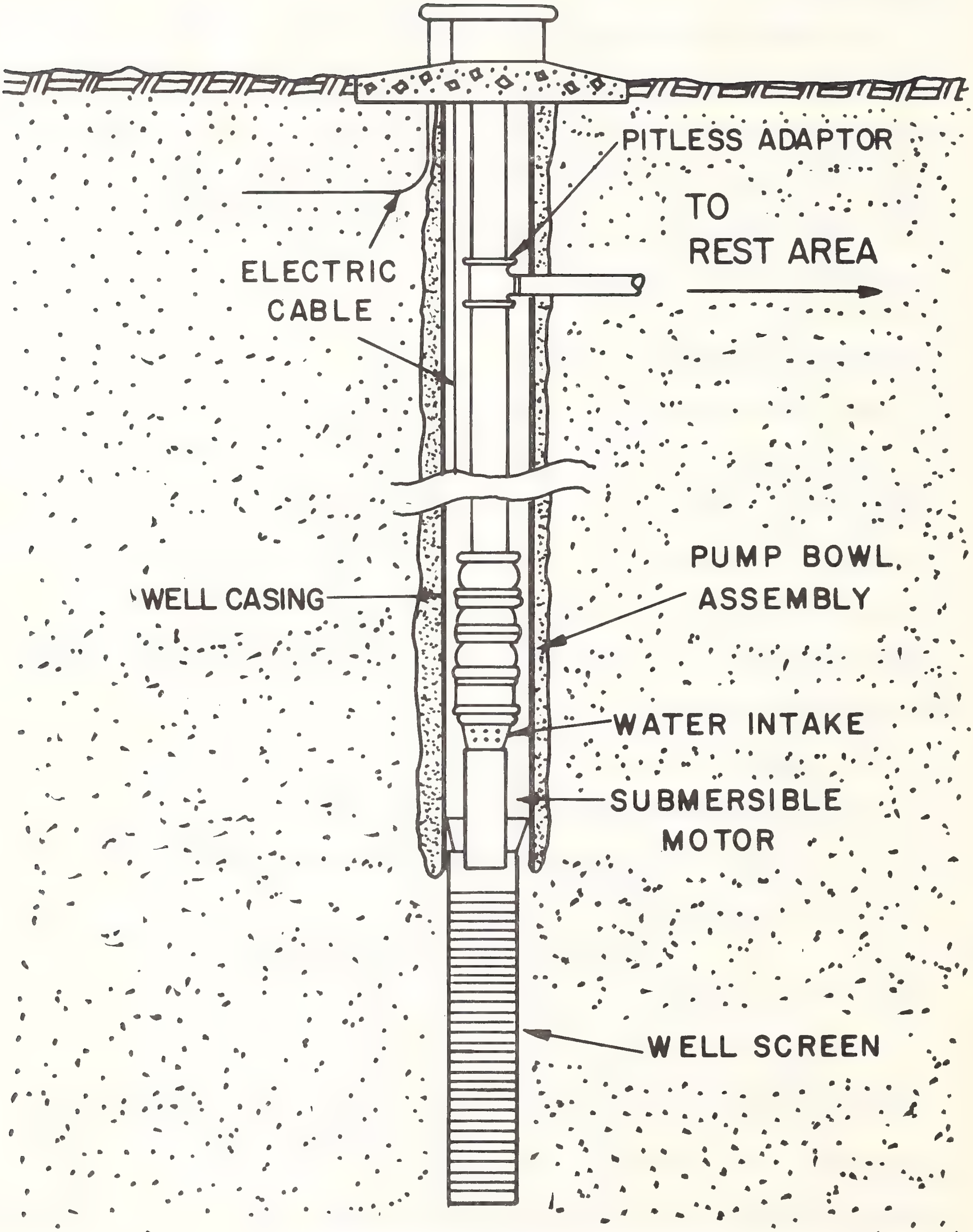
Ho = Atmospheric pressure corresponding to altitude in feet of water. See Table 4.21.

Hv = Saturation vapor pressure corresponding to the water temperature in feet of water. See table 4.22.

h = Height of impeller above water surface

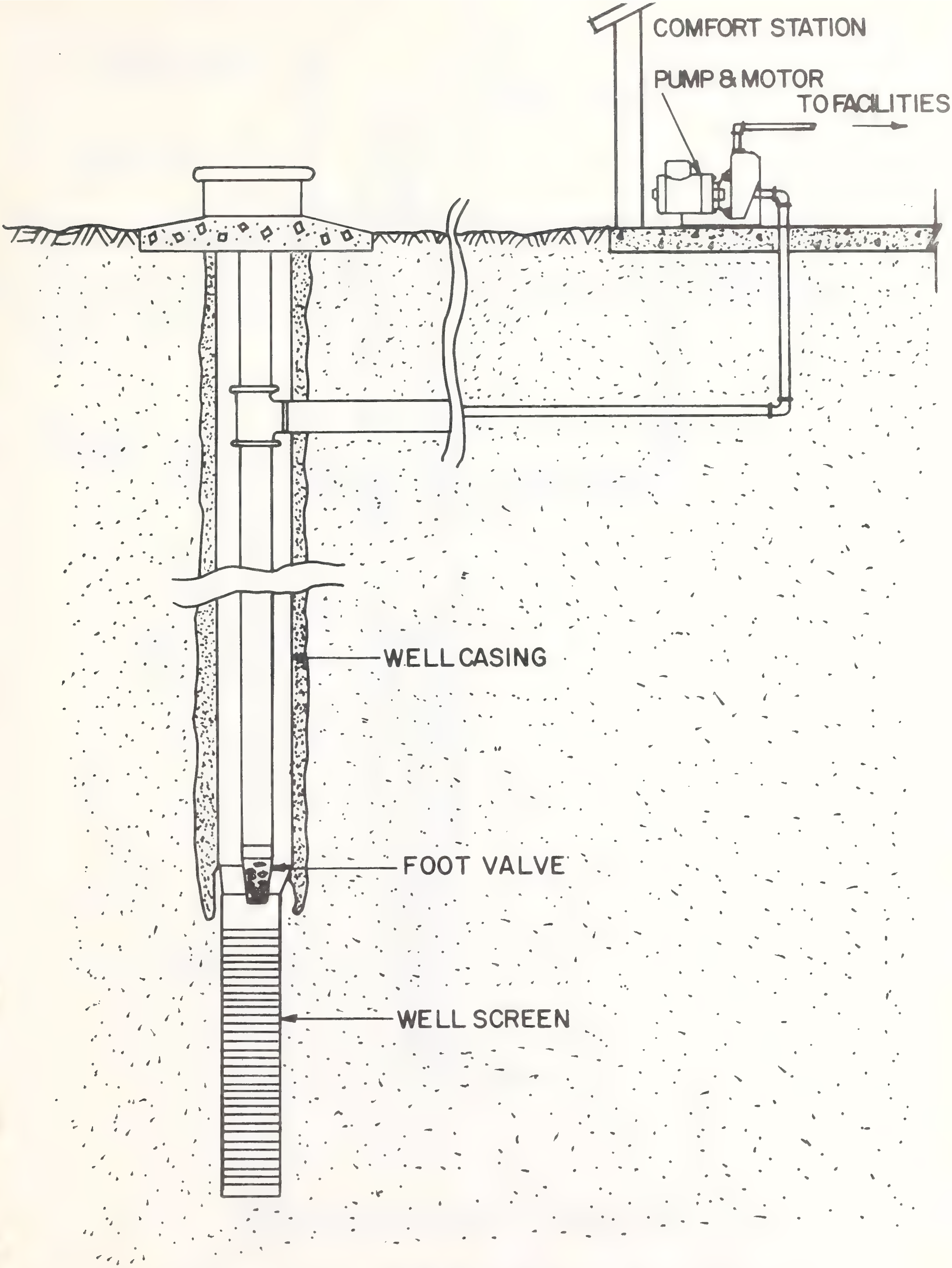
hsl = Losses in suction line in feet

The net positive suction head required (NPSHR) is determined from the performance curve for the specific pump under consideration. NPSHA must be greater than NPSHR or cavatation will occur. Figure 4.32 shows a typical centrifugal or shallow well jet installation.



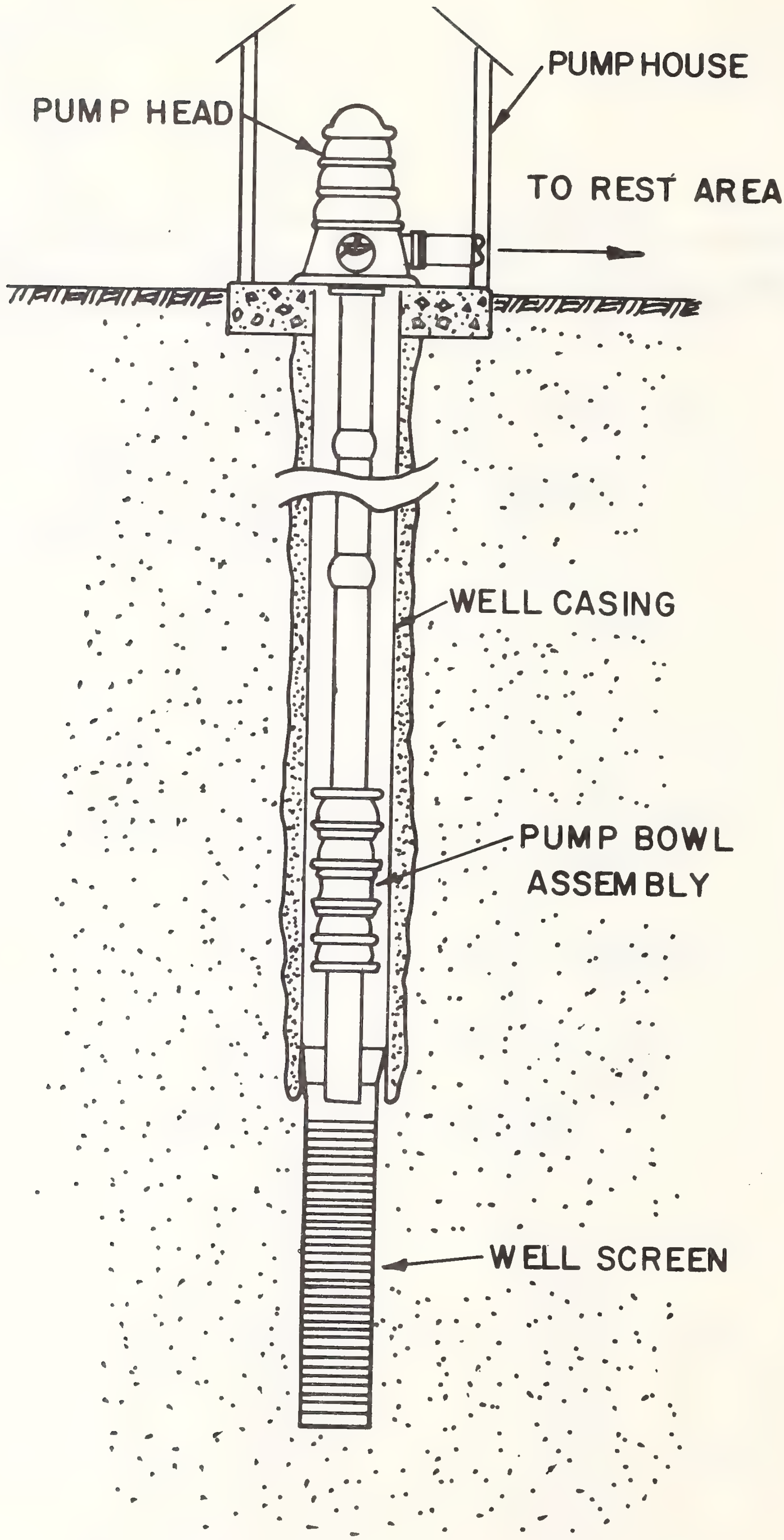
SUBMERSIBLE PUMP

FIGURE 4.31



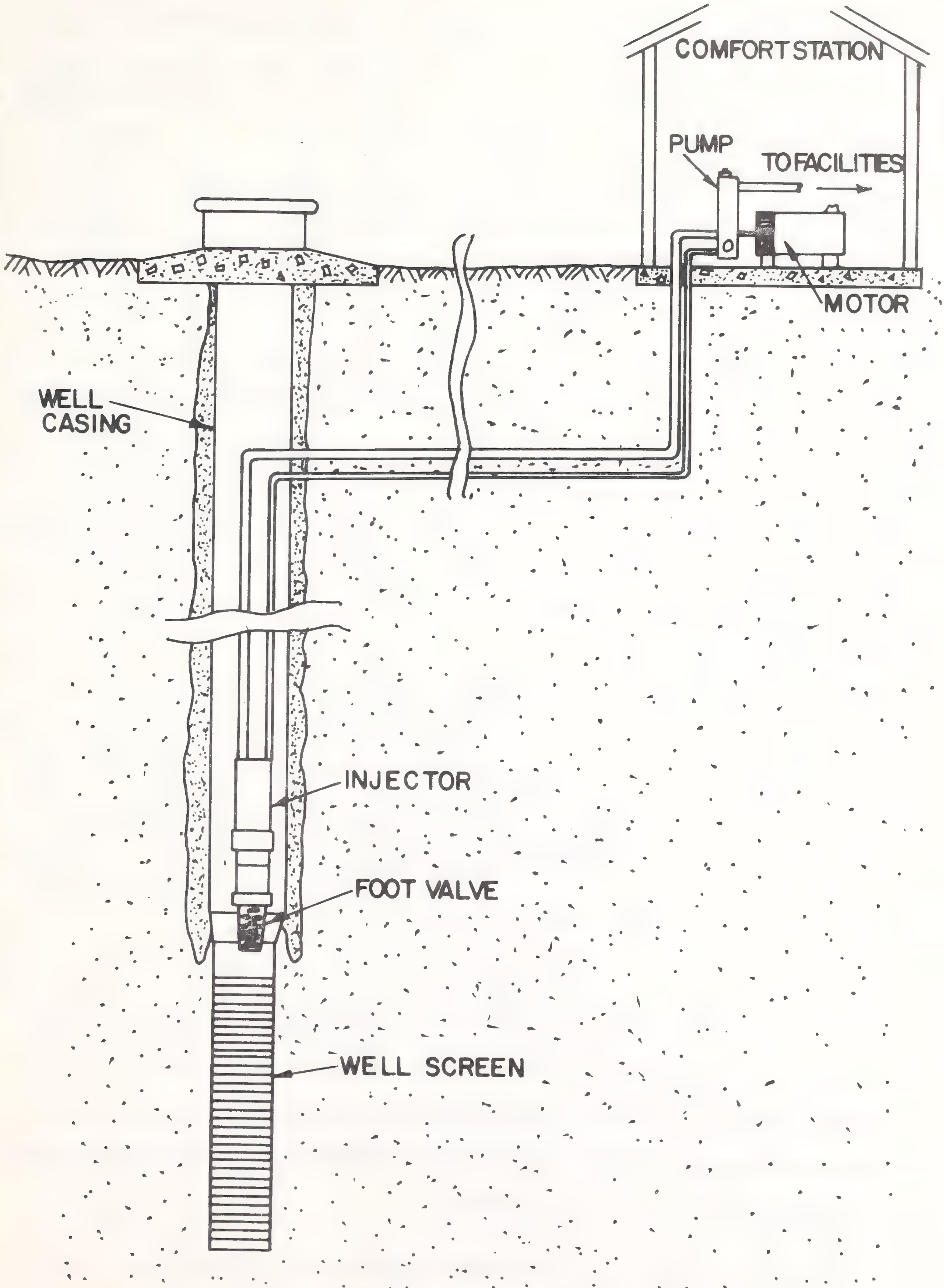
CENTRIFUGAL OR SHALLOW WELL JET PUMP

FIGURE 4.32



LINESHAFT TURBINE PUMP

FIGURE 4.33



DEEP WELL JET

FIGURE 434

Table 4.21

Altitude	Atmospheric Pressure in Ft. of Water
Sea Level	34
2,000	31.7
4,000	29.4
6,000	27.3
8,000	25.2
10,000	23.4
12,000	21.6

Table 4.22

Temperature	Vapor Pressure in Ft. of Water
60°F	Negigible
60	.59
80	1.17
100	2.19
140	6.67
180	17.3
212	34.0

Lineshaft Turbine

The lineshaft turbine is similar to the submersible with the exception that the motor is mounted above the well casing and is connected to the pump, which is down in the well, with a long drive shaft. A pumphouse must be provided to protect the unit from freezing and vandalism. Figure 4.33 shows a typical turbine pump installation.

Deep Well Jets

A deep well jet pump is the same as a shallow well jet pump except the injector is placed down in the well. This elminiates the problem with the suction head and allows the use of this type of pump in deep wells. An additional pipe must be run down into the well to supply the injector. Since the pump does not set down in the well, it must be placed in the comfort station or a pumphouse. Figure 4.34 shows a typical deep well installation.

Pump Specification

The type of pump, flow rate, and total discharge head at the well must be specified to insure that the pumping system provided by the contractor is compatable with the remainder of the water system and capable of supplying the water demand at the desired pressure. These specifications are normally included in the Special Provisions for the project.

The type of pump used in normal installations is the submersible. The flow rate is the peak hourly demand. The discharge head at the well head is the total

of all friction losses and lifts from the well head to the point of use plus the pressure required at the point of use.

Total Pumping Head

In some cases before a pump can be checked for adequacy, the total pumping head must be calculated. The total pumping head consists of all lifts and friction losses plus the discharge pressure. These heads for a submersible pump installation are discussed below and illustrated in Figure 4.35. The losses for other types of pump installations may vary and pump engineering catalogs should be consulted for more information.

The static water level and drawdown can be obtained from Form HYD-2 for wells drilled by highway personnel and from the well logs and test pumping data submitted for wells drilled by a contractor. Drawdown for pumping rates not included in the test pumping data can be determined as in the Well Capacity Section.

The friction losses in the well depend on the pumping rate and the size and type of discharge pipe. Friction factors for various types of pipe can be found in most pump catalogs. Some manufacture's performance curves automatically consider friction losses in the well and care should be taken to insure that they are not considered twice.

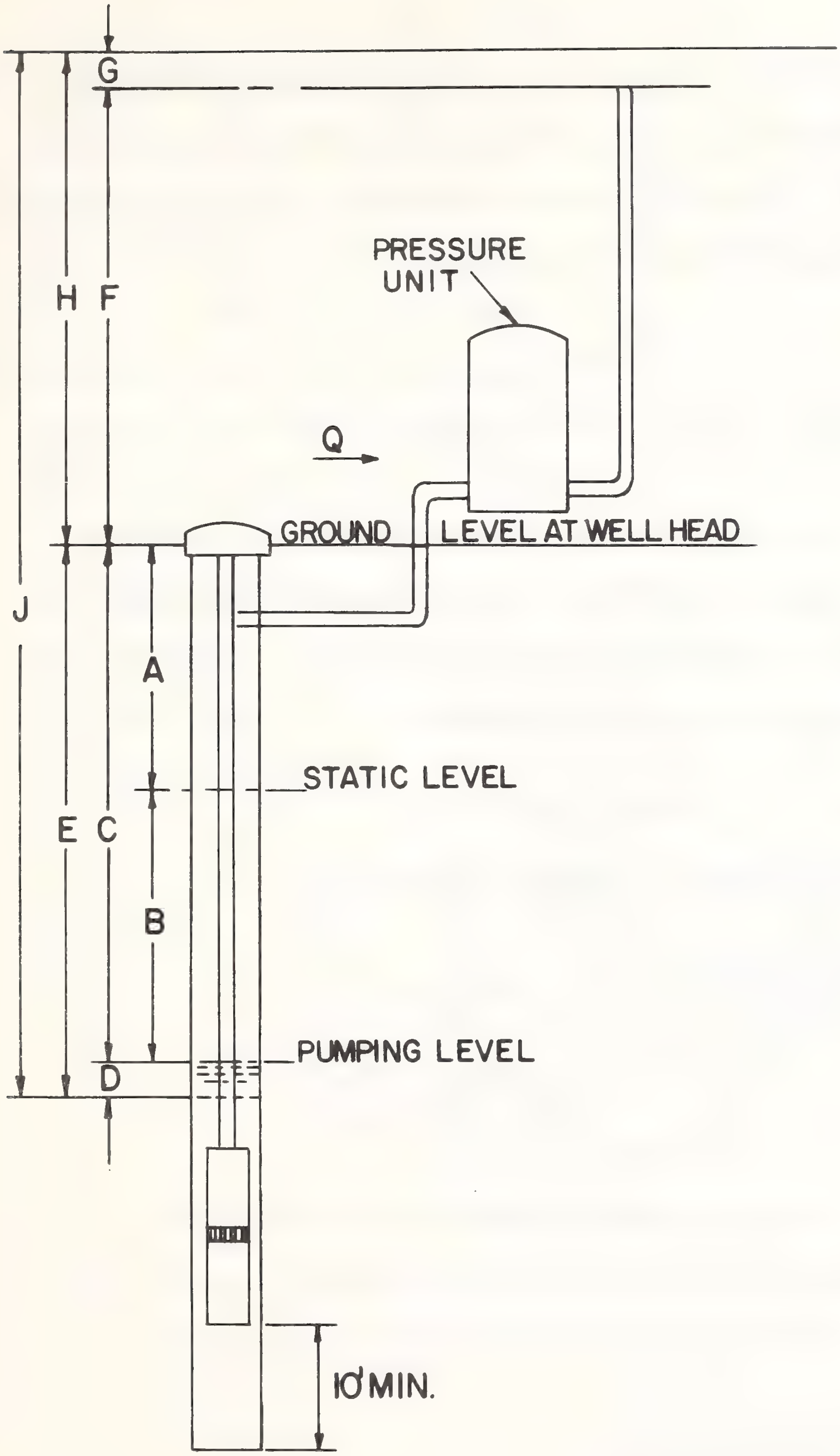
The static discharge head consists of the elevation head between the well head and the discharge point plus the required discharge pressure at that point expressed in feet of head (1 PSI = 2.31 Ft. of head).

The friction loss in the discharge system includes all friction losses in the pipe, elbows, valves and other controls between the well head and the point of discharge.

Well Screen

In some cases a well screen will be required due to sand in the well. In

such cases the Materials Bureau shall be requested to make recommendations pertaining to length of screen and size of slots.



A STATIC WATER LEVEL (feet) ... vertical distance from top of well to natural water level (water table).

B DRAWDOWN (feet) ... reduction in water level during pumping—varies with well yield and pump capacity.

C PUMPING WATER LEVEL or LIFT (feet). C = A + B.

D FRICTION LOSSES in WELL (feet or lb/sq.in.)

E TOTAL LIFT in WELL (feet) ... E = A + B + D

F STATIC DISCHARGE HEAD (feet) ... sprinkler system - elevation of highest sprinkler head above well head plus pressure required at that head. rest area - elevation at pressure unit above well head plus pressure required at pressure unit.

G FRICTION LOSSES in DISCHARGE SYSTEM ... in pipe and fittings between well head and point of discharge.

H TOTAL DISCHARGE HEAD (feet) ... H = F + G.

J TOTAL PUMPING HEAD (feet) ... J = E + H.

Q CAPACITY (gpm-gallons per minute) demand rate.

SCHEMATIC LAYOUT USING SUBMERSIBLE

FIGURE 4.35

4.32 PRESSURE SYSTEM

Design of the pressure system consists of determining the maximum allowable cycling rate for the pump, calculating the draw-off required to maintain this cycling rate, and specifying the size and type of pressure unit(s) and other materials to be used to complete the system.

Cycling Rate

Frequent switching on and off of the pump can cause the motor to overheat, shorten the motor's life, and require more energy. To reduce the operating and maintenance cost, the cycling rate for the pump shall be established based on the size of the motor as shown in Table 4.23. The test pumping data, water demand, and the manufacturer's performance curves can be used to determine the approximate horsepower of the pump that will be used.

Table 4.23 Maximum Cycling Rate

H.P. Rating of Motor	Maximum Cycles/Hour
1/4 through 2	30
3 and 5	20
7 1/2 through 50	10
60 and larger	6

Draw-Off

To insure that the cycling rate remains below the maximum allowable, pressure units will be required that provide sufficient usable storage. This storage is called draw-off and is calculated by the following formula:

$$\text{Draw-Off} = \frac{\text{Water Demand} \times 15}{\text{Maximum cycling rate}}$$

Where Draw-Off = The required usable storage in gallons

Water Demand = The flow rate required by the sprinkler system or the comfort station facilities whichever is greater in gallons per minute

Maximum cycling rate = Maximum cycling rate from Table 4.23 in cycles per hour

Pressure Units

The usable storage capacity for draw-off calculated above shall be provided by a pressure unit or units. These units shall be compact and shall contain a membrane to separate the water from air to eliminate water-logging. The units specified must also be capable of being connected in series to provide the total required draw-off and work as a single unit. There are only two products known that meet these requirements: (1) The Hydrocel manufactured by Jacuzzi Bros., Inc. and (2) The Well-X-Trol manufactured by American Tube and Controls.

Tables 4.24 and 4.25 give the draw-off capacities for the various models of the Hydrocel and Well-X-Trol, respectively. These tables are based on a pump cut-in pressure of 20 psi and pump cut-out pressure of 50 psi. A pressure range of 20 psi to 50 psi is recommended for rest areas. Manufacturer's design brochures should be consulted for pressure ranges other than 20 - 50 psi.

Table 4.24 Draw-off Capacities for Hydrocells

Hydrocell Model	Draw-off Capacity per Hydrocell
Blue (9190-7014)	2.0 gal.
Green (9190-7022)	1.84
Jumbo (9190-7048)	3.30

Table 4.25 Draw-off Capacities for Well-X-Trol

Well-X-Trol Model	Draw-off Capacity per Unit
WX-101	.92 Gal.
WX-102	2.12
WX-103	3.60
WX-201	6.44
WX-202	8.28
WX-203	14.0
WX-204	20.0
WX-301	69.0
WX-302	92.0
WX-303	115.0
WX-304	142.6
WX-305	150.0

Other Pressurizing Products

There are other products on the market that will do the job of the pressure system but they are not acceptable for rest area use. Some of these products will be discussed briefly so the designer will be familiar with them.

The standard hydro-pneumatic pressure tanks contain a cushion of air which expands and contracts to pressurize the water and provide storage. The greatest problem with the standard pressure tank is absorption of the air by the water with time. This condition, known as water-logging, will make the tank useless until it is recharged with air. It is the maintenance associated with this recharging that makes the standard hydro-pneumatic pressure tank unacceptable for rest area installation. The standard pressure tank also requires alot more room than the other pressurizing systems.

There are several accessories available which when used with the standard hydro-pnuematic pressure tank improved the system's characteristics. One is the automatic air charger which maintains the right amount of air in the tank at all times and thus eliminates water-logging. However, the automatic air charger can only be used with centrifugal and jet pumps and the tank must be located near the pump. Another is a float that is placed inside the tank and

floats up and down with the water. The float separates the water and air and slows down the water-logging process. However, it does not completely eliminate water-logging.

Another product which will serve as the pressure system is the Aqua Genie manufactured by Jacuzzi Bros., Inc. The Aqua Genie consists of a pressure switch, a special valve, and a minature Hydrocel. The instant water is used anywhere in the system, the Aqua Genie turns the pump on and leaves it on until the use stops. This can reduce cycling in some types of water systems and increases it in others. It is felt that the Aqua Genie would increase cycling in a rest area installation because of the nature of the water use. However, the Aqua Genie would probably provide a very adequate pressure system for a weigh station. The Aqua Genie takes much less space than any other type of pressure system.

4.33 PURIFICATION SYSTEM

Because a rest area water supply offers an unparalleled ability to transmit communicable water-borne diseases across the Nation, it is important to maintain a safe and high quality water supply. The quality of water is determined by its physical, chemical, biological, and radiological characteristics.

The physical characteristics include the water's turbidity, color, taste, odor, and temperature, while the chemical characteristics include toxicity, mineral content, aklalinity, hardness, and pH. The biological characteristics include the bacteria content. The water must be free of pathogenic bacteria.

The water quality tests and correspondence from the Materials Bureau will indicate any unacceptable conditions and make recommendations for their correction if the water is to be used for drinking.

The rest area should be plumbed so only the water going to the drinking fountains and lavatories is purified, when purification is necessary. This will greatly reduce the size of the purification unit required.

The purification system should be large enough to handle the peak hourly flow to the lavatories and drinking fountain. The flow rate can be calculated from the following formula:

$$Q_p = ADT \times DF \times S \times P \times SPK \times K \times D \times G \times G_p \times \frac{1}{60}$$

ADT = Estimated Average Daily Traffic for the design year

DF = Direction Factor

1.0 = if both directions

.5 = if only one direction

S = Percent of vehicles stopping at rest area per day (5% to 14%)

P = Number of people per vehicle using comfort station facilities (usually 2.25)

SPK = Seasonal peak factor (usually 1.30)

K = Factor representing portion of daily flow occurring during peak hour. See Figure 4.31 (usually 0.15)

D = Direction distribution allowance

1.0 = if both directions served

1.2 = if only one direction served.

Gp = No. of gallons per person for drinking fountain and lavatories (usually 0.5)

Qp = Peak flow in GPM that must be purified

There are four types of purification systems that should be considered for rest area use; chlorinators, iodinators, ultraviolet sterilizers, and reverse osmosis systems. Each system has advantages and disadvantages. The system used will depend on the water condition that is being corrected, installation costs, maintenance costs, space available and the recommendations from the Materials Section. A description of these four types of systems follows:

Chlorinators

Chlorine can be used to destroy bacteria, slime and algae and eliminate taste and odor in drinking water supplies. The typical treatment rates for ground water supplies are .5 parts per million (PPM) for the destruction of bacteria, algae and slime and approximately 2 PPM for taste and odor elimination. The control of odor and taste by the use of chlorine is quite complex and a good water treatment text should be consulted before using chlorine for this purpose. Chlorine used for water purification may be either chlorine gas or hypochloride, depending on the type of unit used. Chlorine gas is sold in steel cylinders and can usually be purchased from municipal water departments or chemical companies. Common household bleach is a typical hypochloride and may be purchased in most food stores. Chlorine gas is the more economical form, but since the amount of chlorine used in rest area applications is minimal, this is not an important consideration. Chlorine gas is very toxic and can be dangerous. It must be handled with great care under adequate safeguards.

Chlorine does not kill bacteria instantly upon contact; therefore, the chlorine must remain in contact with the water before use until the desired kill is obtained. A contact period of 15 minutes is usually required for the desired kill. A holding tank with a volume at least equal to the contact time in minutes times the flow rate in gallons per minute calculated previously will provide sufficient contact time.

The specifications for the chlorinator should state that the chlorinator provided be capable of supplying twice the anticapated required dosage at the peak flow rate calculated above. The system must also be capable of maintaining a proportionate feed regardless of fluctuating water demands.

Iodinators

Iodine has recently become an acceptable disinfectant for use in small purification systems and is in many ways superior to chlorine. Iodine will destroy bacteria, viruses, fungi and amoebic cysts and eliminate hydrogen sulfide odors. The recommended dosage for ground water supplies is 0.5 PPM.

Iodine, like chlorine, requires contact time of 15 minutes to obtain the desired kill. The volume of the required holding tank can be determined as explained for chlorinators.

The advantages of iodinator include low initial costs and very low maintenance costs. One pound of iodine crystals can be used to disinfect approximately 250,000 gallons of water at 0.5 PPM.

Ultraviolet Light

Ultraviolet light can also be used as a disinfectant to kill bacteria and viruses. The water is fed into a quartz water jacket where it is exposed to a high intensity ultraviolet light which kills the pathogenic organisms.

The main advantage of the ultraviolet process is the small amount of space it requires. This is because no holding tank is required. Because the

ultraviolet process does not add any chemicals to the water there are no changes to the chemical and physical characteristics.

There are several disadvantages of the ultraviolet system. Initial and maintenance costs are higher than for the previously discussed systems. Color, turbidity, and organic impurities interfere with the transmission of ultraviolet energy and may decrease the disinfection efficiency below levels required to insure destruction of the pathogenic organisms. Since the ultraviolet system only disinfects within the water jacket, it is necessary to periodically flush and disinfect the water distribution system.

Reverse Osmosis

The reverse osmosis system can be used in the more complicated purification situations. The reverse osmosis system can be used to remove any of the following minerals: calcium, magnesium, sodium, potassium, manganese, aluminium, seilica, bicarbonate sulfate, chloride, nitrate fluoride, boron (as anion), orthophosphate and polyphosphate. The system will also remove organic matter, pyrogens and bacteria.

The reverse osmosis system works on the principle of movement of substances from concentrated solutions to dilute solutions. The contaminated water is pressurized and forced through a semi-permeable membrane which prevents the passage of contaminates.

The actual design of the reverse osmosis system depends on the chemistry and characteristics of the water being treated. For that reason the design is left up to the manufacturer.

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SECTION 4.4
SPRINKLER SYSTEMS

4.4 SPRINKLER SYSTEMS

Introduction

The design of an underground sprinkler system involves arranging sprinklers, piping, and controls together into a system that best fits the conditions of the area and produces the desired turf and plant results. The sprinklers, piping, controls and pumping system must be sized so a balanced and efficient system is achieved. A poorly designed system will not do the job for which it is intended. Unsightly dry spots will occur, shrubs and trees may die, and maintenance costs will skyrocket. A little time, effort, and expense to see that a good, balanced system is designed will pay high dividends in the form of beautifully landscaped rest areas and interchanges and low maintenance costs.

In order to properly design a sprinkler system it is necessary to follow a procedure, such as the one listed below and described on the following pages, to insure that the many variables are considered.

1. Obtain Site Data
2. Select Sprinkler Types and Design Characteristics
3. Locate Sprinkler Heads
4. Divide System into Circuits
5. Route Circuit Piping and Size Pipe
6. Provide for Miscellaneous Systems

The design manuals published by sprinkler equipment manfacturers should be consulted if more information or guidelines are needed.

4.41 OBTAIN SITE DATA

It is very important to secure complete and accurate field information of the actual site that is to be watered. Without a complete and accurate plan of the field conditions, a system cannot be designed without making assumptions about the site. Some of the major information required is listed in the following paragraphs. Most of this information can be obtained from the Landscape Unit or from the Location and Road Design Section.

Plot Plan - The plot plan should be a scaled layout of the site to be landscaped. One inch equals ten feet (1"=10') or one inch equals twenty feet (1"=20') are usually the most convenient scales to work with. The plan should include the exact location and size of the comfort station or buildings as well as all sidewalks, curbs, roadways, picnic benches, parking areas, etc. which will affect the design. Also indicate where sprinkler over throw is permissable.

Type of Plantings - The plot plan should show all areas that will be seeded, all areas that will be sodded, all areas where natural vegetation will be used, and indicate which of these areas will be watered and which will not. The plan should also show all shrubs, bushes, trees, and flower beds, and note which of these plantings should be watered and whether they should be watered with a bubbler or a shrub spray. Also indicate the affect of these plantings on turf sprays placed near them.

Elevation Changes on the Site - Any changes in elevation that occur on the site should be shown on the plan. These elevation changes can be shown best with a contour map of the site. Elevation changes are necessary to compute pressure gains and losses throughout the system and to estimate the number of drain valves required.

Prevailing Wind Direction and Speed - Prevailing wind velocity and direction are required to determine the maximum spacing of sprinklers to assure proper coverage. The wind may also have some influence on the type of sprinklers that may be used.

Local maintenance personnel should be able to provide this information.

<u>Type and Source of Water</u> - The location of the water source and the type must be known for each sprinkler system. If the source is a well, the well capacity must also be known. If the source is a canal or stream, the amount of water available, the location of the intake and the water surface elevation at the intake must all be known. If city water is to be used the location and size of the nearest city water main, the pressure and amount of water that may be used must all be known.

<u>Miscellaneous Information</u> - The following information must also be known; the location of the nearest power source, the type of service (single or three phase, 120 or 240 volts), the preferred location of the controllers if they are to be used, and the type of system (manual or automatic) that is preferred by the personnel who have to maintain it.

4.42 SELECT SPRINKLER TYPES AND DESIGN CHARACTERISTICS

There are several basic types of sprinkler heads, each having different characteristics and uses. Each type of head is described below and a short discussion of its uses follows.

Bubbler Heads - A bubbler head is one which provides flood type irrigation and is used primarily for shrubs, trees, and flower beds. Bubblers should be fully adjustable with flow rates ranging from 0 to 2 gallons per minute (GPM). The preferred operating pressures are below 20 psi.

Turf Spray Heads - Turf spray heads are used to water small turf areas. They are installed flush with the ground and have a stem that pops up during operation They operate in the low pressure range of 15 to 35 psi and apply water at the high rate of application of 1 inch to 2 inches per hour. Their radii range of 10 feet to 20 feet makes them most economical for small and irregular shaped areas.

Shrub Spray Heads - The shrub spray head is similar to the turf spray head except it is usually mounted on a riser pipe and does not have the pop-up stem. It is used for watering shrubs, bushes, ground cover, and flower beds. Because the spray head sets above the vegetation on riser pipe, the vegetation cannot break up the spray distribution. The pressure range for shrub sprays is 15 to 30 psi. They apply water at a relatively high rate with radii ranging from 7 feet to 15 feet.

Steep Slope Spray Heads - The steep slope spray head is a shrub spray with its head designed for low precipitation rates. This head can be used on steep slopes like highway embankments without causing erosion. The recommended pressure range for these heads is 15 to 30 psi with radii up to 22 feet.

Pop-Up Impact Heads - The pop-up impact heads are the most economical heads for use in large open turf areas. They operate at high pressures (30 psi to 80 psi)

and cover large areas (40 feet to 100 feet radius). They apply water at a low rate which is generally from .2 inches to .5 inches per hour. The pop-up impact head is the most widely used head for larger areas because of the wide selection of heads and characteristics.

Pop-Up Cam Driven Heads - The pop-up cam drives have characteristics very similar to pop-up impact heads, i.e., pressure ranges from 30 to 80 psi, radii from 30 feet to 80 feet and low precipitation rates. The cam drive is most economical in large open turf areas and is especially good for borders because there is no back splash. There is not a good selection of pop-up cam driven heads and this limits their use. An internal cam is used to rotate the head.

Pop-up Gear Driven Heads - The pop-up gear driven heads have uses and characteristics similar to the impact and cam driven heads. Since some manufacturers do not make a gear drive and there is not a good selection from those who do, the gear drive is not used much.

The first step in determining the design sprinkler characteristics is the selection of the type or types of heads to be used. The plot plan should be reviewed and the type of sprinkler heads to be used for the various plantings should be decided. Generally, turf spray heads are used for small irregular turf areas, pop-up rotor heads are used for large open turf areas, steep slope spray heads are used when slopes exceed 4:1 and bubblers or shrub sprays are used for all trees, bushes, shrubs, and flower beds.

Once the type of heads to be used for the various plantings has been selected, it is necessary to determine the characteristics (flow pressure and coverage radius) that will be used in design. Due to Federal regulations on propriatory items, the characteristics selected and specified must be such that more than one manufacturer can supply the sprinkler heads. Therefore, several manufacturer's performance charts must be reviewed before specifying flow rate, operating pressure, and radius of coverage.

The design flow rate, radii, and pressure should be recorded as they will be required to write the specifications for the sprinkler heads. The model number of each manufacturer's head should also be recorded as they will also be included in the specifications.

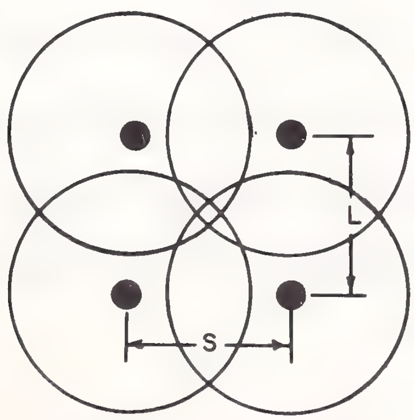
4.43 LOCATE SPRINKLER HEADS

Once the design characteristics for each type of head to be used are determined, it is necessary to locate the heads. This is done on the scaled plot plan.

All trees and some bushes, shrubs, and flower beds are best watered with bubbler heads. Generally, one bubbler is used for each tree, bush or shrub. However, if two such plantings are within five feet of each other, one bubbler may be used to water both.

Bushes and shrubs placed in groups or hedges and most flower beds are best watered with shrub spray. Because shrub spray heads are set on risers, their spray patterns are more susceptable to distortion by wind than other heads. Therefore, spacing between shrub spray heads should not exceed the design radius determined in Section 4.42.

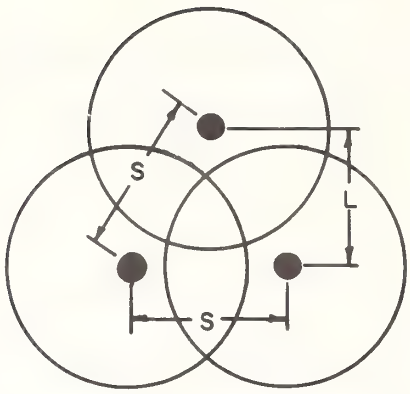
The spacing of sprinklers for turf areas depends on wind conditions in the vicinity where the sprinkler system will be placed. Wind destroys the sprinklers distribution pattern, therefore, if wind is expected, the sprinklers must be spaced closer together to insure adequate coverage. Lateral line spacing depends upon the pattern which is used to layout the system. There are two basic patterns used which are illustrated below.



SQUARE SPACING

L = Spacing between lateral lines = S

S = Spacing between sprinklers



TRIANGULAR SPACING

S = Spacing between sprinklers

L = Spacing between lateral lines - .86 x S

Table 4.26 gives the recommended spacing (S) in percent of diameter of sprinkler coverage for different combinations of wind velocity and spacing pattern.

Table 4.26
Recommended Sprinkler Spacing

Spacing Pattern	Wind	Spacing
Square	No Wind	55% of Diameter
Square	4 MPH (Average)	50% of Diameter
Square	8 MPH	45% of Diameter
Triangular	No Wind	60% of Diameter
Triangular	4 MPH (Average)	55% of Diameter
Triangular	8 MPH	50% of Diameter

Because most of the turf areas that must be watered are irregular, it is often impossible to use either a square or triangle spacing pattern exclusively. The triangular pattern is preferred over the square pattern but a combination of the two is usually required.

4.44 DIVIDE SYSTEM INTO CIRCUITS (Sectioning)

Once the location of all the heads is shown on the plot plan it is necessary to divide the heads into circuits.

Every head sectioned on an individual circuit will operate together and will have the same operating pressure, plus or minus friction and elevation losses. Since these heads will be operating together, it is important to section heads so that all the heads on a circuit require nearly equal operating pressures and have nearly equal precipitation rates. In many instances it will be impractical or impossible to section the heads as desired due to the number of a particular type head used and the remote locations of one or two heads. By following the procedures and rules of thumb discussed below, a design that is a compromise between the most efficient and most economical can be achieved.

1. The design water demand for most rest areas with sprinkler systems and all interchanges is established by the water requirements for the sprinkler system and is generally supplied by a pump. To insure that the pump operates at maximum efficiency at all times, the water demands for each circuit should be nearly equal and the pump sized accordingly.

This can be accomplished by dividing the sum of the demands of all heads by the approximate capacity of the well to determine the approximate number of circuits and adjusting to a higher integral valve. For example:

> A rectangular field requires 24 full circle spray heads at 3.8 g.p.m. each and 18 half circle spray heads at 2.5 gpm each. The capacity of well = 20 gpm. No. circuits = $\frac{(24 \times 3.8) + (18 \times 2.5)}{20} = \frac{136.2}{20} =$ 6.8 Circuits.

This could be sectioned in several ways, three of which are listed below.

A. Three (3) circuits with six (6) half circle heads - 15.0 GPM Each and six (6) circuits with four (4) full circle heads - 15.2 GPM Each

B. Two (2) circuits with eight (8) half circle heads - 20.0 GPM Each and four (4) circuits with five (5) full circle heads - 19.0 GPM Each and one (1) circuit with four (4) full circle heads and two (2) half circle heads - 20.2 GPM Each.

C. Four (4) circuits with three (3) full and two (2) half circle heads - 16.4 Each
and two (2) circuits with two (2) full and four (4) half circle heads - 17.6 GPM Each
and two (2) circuits with four (4) full and one (1) half circle heads - 17.7 GPM Each

2. Because impact heads operate at much higher pressures than fixed spray or bubbler heads, they should never be combined on a circuit.

3. If at all possible bubblers should be placed on circuits by themselves but they can be used with spray heads if necessary.

4. The water applied by most impact heads is dependent only on the water pressure and nozzle size. Therefore, a part circle head will apply water at a higher precipitation rate than the same head while operating as a full circle sprinkler. Combining such heads on the same circuit should be avoided whenever possible. When it is necessary to combine such heads to have a balanced system and to reduce trenching and pipe lengths, heads with the more nearly equal precipitation rates should be combined first.

5. Whenever possible, sprinkler heads should not be combined on circuits if an elevation head exists between the sprinkler heads that is greater than 10% of the operating pressure.

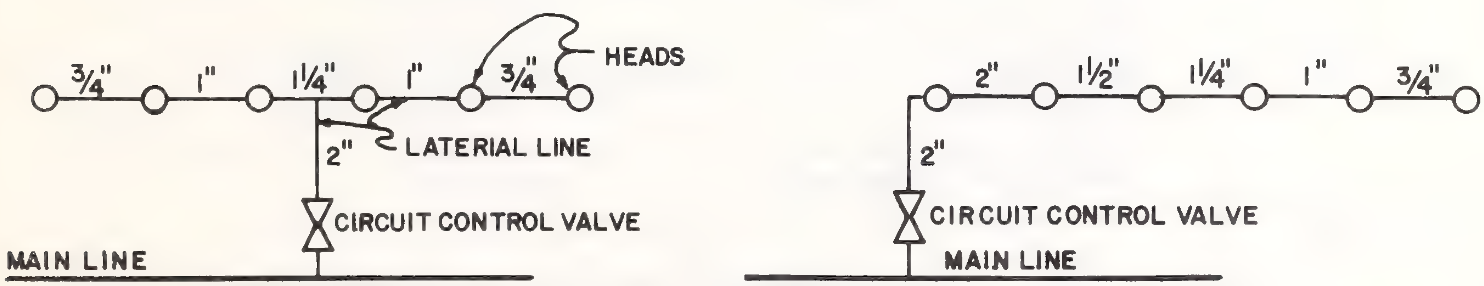
6. Sectioning heads involves a trial and error process to attain a nearly balanced system with a minimum of extra trenches or pipes.

4.45 ROUTE CIRCUIT PIPING AND SIZE PIPE

At the present time, PVC (Polyvinyl Chloride) pipe is the most suitable for sprinkler system use. Class 160 and 200 are normally used for sprinkler system designs, where the 160 and 200 denote the maximum working pressure of the pipe. For pipe one inch in diameter and smaller, Class 160 pipe is actually the same as Class 200 pipe. Class 200 pipe should be specified for any line that is under pressure throughout the entire sprinkling season and for all lines on systems where the water carries sand or grit.

There are two types of lines to be sized for a sprinkler system, (1) Main lines and (2) lateral lines. The main lines are the feeder lines extending from the pump or water supply system to the control valves. These lines are under pressure whenever any circuit of the system is operating. The lateral lines are those lines connecting the individual heads within a circuit and the circuit control valve. Lateral lines are pressurized only when that circuit is operating. Both the main lines and the lateral lines shall be shown on the plot plan.

When connecting the main line to a lateral line, it is preferred that the connection be made near the lateral mid point with branching from that point rather than have a long single series. The following diagrams show these methods and illustrates the component parts:



BRANCHING METHOD (PREFERRED)

SERIES METHOD (UNDESIRABLE)

Although the branching may sometimes require slightly more pipe, it allows the use of smaller pipe within the circuit and reduces the pressure variation

between heads.

The major consideration in sizing pipe for a balanced system is assuring that the variation in pressure between any two heads on a circuit is less than 20% of their operating pressure. Also, for a balanced system, the pressure loss between the water source and the last sprinkler head on a circuit must be nearly equal for each circuit with the same type heads. These pressure losses include friction losses, elevation losses (plus or minus), and losses through valves, but does not include losses through the heads.

The friction factor method shall be used to determine pipe sizes. The allowable friction factor for each circuit is calculated by the following formula:

$$F_f \text{ (Friction Factor)} = \frac{\text{Allowable loss in psi}}{\text{Total Length of Pipe in Circuit/100}}$$

where

Allowable loss = 20% of the operating pressure of the heads plus estimate of allowable loss in lead-in line.

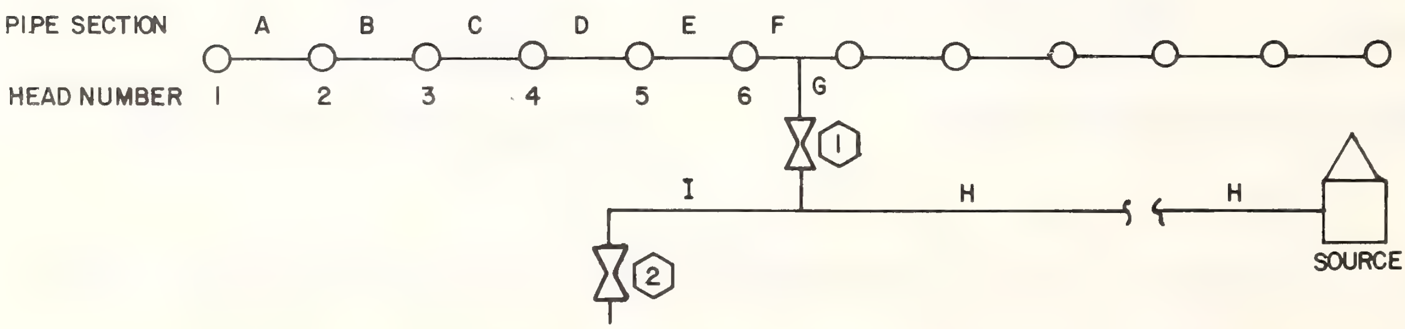
Total length of pipe = length between the valve and the last head on the circuit.

The following example is given to illustrate the steps to be followed in this method. This method is solved by computer program "SPR Basic".

Example:

Data:

Type of heads	=	Spray Heads
Operating Pressure	=	30 psi
Type of Pipe	=	PVC Class 200
Discharge	=	3 GPM per head
Head Spacing	=	15 Feet
Pressure at Source	=	50 psi
Elevation Differences	=	Negligible



In the above example, the friction losses on the left will be the same as those on the right; therefore, the allowable loss is equal to the loss between heads 1 and 6. The length of pipe between the valve and head 6 is included in the circuit length for calculating the friction factor to reduce the design steps. The allowable loss is then selected as 9 psi (.20 x 30 plus 3 psi allowance for section G). $F_f = \frac{9}{208/100} = 4.33$ psi/100 Ft. Using Table 4.29 for Class 200 PVC pipe, select the size of pipe necessary to maintain an actual loss/100 feet less than F_f for each section of pipe. The actual loss for each section of pipe and the total loss for the circuit is calculated. The Manual procedure is illustrated in Table 4.27. The friction loss between heads 1 and 6 can be checked by adding the losses for sections A, B, C, D, and E. The loss is 1.42 psi, well below the maximum of 20% of the operating pressure.

The final considerations in balancing the system are selecting the control, sizing the main lines, and determing whether a pressure reducing valve is required. The final determinations for each of these are inter-dependent in addition to being dependent upon the individual circuit requirements, main line lengths, and pressure capabilities of the water source.

Due to these inter-dependencies, specific procedures for design cannot be given here. Instead, some of the more important design considerations are listed below.

1. Velocities in all lines shall be less than 6 ft./sec. to eliminate water hammer.

2. Control valves shall be specified on the plans by allowable pressure loss at the design flow rather than by size.

3. The most efficient system is one having the least possible friction loss; therefore, line losses and losses through valves should be kept low except when necessary to balance the system.

TABLE 4.27

Circuit No. 1		PIPE SIZING CALCULATIONS				Pipe Material: PVC Class 200 Friction Factor: 4.33 psi per 100' of Pipe	
Pipe Section	Flow GPM	Pipe Size Inch	Friction Factor	Velocity	Actual Length Feet	Calculation of Actual Loss	Actual Friction Loss
A	3	1/2"	1.61		15	.15 x 1.61 =	.24 psi
B	6	3/4"	1.67		15	.15 x 1.67 =	.25 psi
C	9	3/4"	3.57		15	.15 x 3.57 =	.54 psi
D	12	1"	1.83		15	.15 x 1.83 =	.27 psi
E	15	1"	2.68		15	.15 x 2.68 =	.40 psi
F	18	1"	3.82		8	.08 x 3.82 =	.31 psi
G	36	1 1/2"	2.30		120	1.20 x 2.30 =	2.67 psi
						SUB-TOTAL	4.77 psi
Valve							6.2 psi
						TOTAL CIRCUIT LOSS	10.97 psi
H	36	2"	0.71		500	5.00 x 0.71 =	3.55 psi
						TOTAL LOSS	14.52 psi

TABLE 4.28

Friction Loss in Class 160 PVC Pipe
PSI per 100 Feet of Pipe C=150

GPM	1/2"	3/4"	1"	1 1/4"	1 1/2"	2"	2 1/2"
1	.21	.06	.02				
2	.76	.22	.06				
3	1.61	.46	.14	.04			
4	2.74	.79	.23	.07	.04		
5	4.14	1.19	.35	.10	.07		
6	5.80	1.67	.49	.14	.08		
7	7.84	2.26	.67	.21	.10		
8	9.87	2.84	.84	.24	.13	.04	
9	12.39	3.57	1.06	.30	.17	.06	
10	14.91	4.29	1.27	.37	.20	.07	.03
11		5.25	1.55	.45	.24	.08	.03
12		6.21	1.83	.53	.28	.10	.04
13		7.17	2.11	.61	.32	.11	.04
14		8.13	2.39	.69	.36	.13	.05
15		9.08	2.68	.78	.41	.14	.05
16		10.35	3.06	.89	.47	.16	.06
17		11.62	3.44	1.00	.53	.18	.07
18		12.89	3.82	1.11	.59	.20	.07
19		14.16	4.20	1.22	.65	.22	.08
20		15.46	4.57	1.33	.70	.24	.09
22			5.04	1.61	.84	.29	.11
24			5.98	1.89	.98	.34	.12
26			7.45	2.17	1.15	.39	.16
28			8.55	2.49	1.33	.45	.18
30			9.67	2.81	1.49	.50	.20
32				3.19	1.69	.56	.23
34				3.57	1.89	.63	.26
36				3.94	2.09	.71	.29
38				4.37	2.31	.79	.32
40				**4.79**	2.54	.86	.34
42					2.78	.94	.37
44					3.02	1.02	.41
46					3.31	1.11	.45
48					3.57	1.20	.48
50					3.84	1.29	.51
52						1.40	.55
54						1.50	.59
56						1.60	.63
58						1.70	.67
60						1.81	.72

Note: Valves below dotted lines are at velocities over 6 feet per second and should be selected with caution

TABLE 4.29

Friction Loss in Class 200 PVC Pipe
PSI per 100 Feet of Pipe C=150

GPM	1/2"	3/4"	1"	1 1/4"	1 1/2"	2"	2 1/2"
1	.21	.06	.02				
2	.76	.22	.06				
3	1.61	.46	.14	.04			
4	2.74	.79	.23	.08	.04		
5	4.14	1.19	.35	.12	.06		
6	5.80	1.67	.49	.16	.08		
7	7.84	2.26	.67	.22	.11		
8	9.87	2.84	.84	.28	.14	.05	
9	12.39	3.57	1.06	.35	.18	.06	
10	14.91	4.29	1.27	.42	.21	.07	.03
11		5.25	1.55	.51	.26	.09	.03
12		6.21	1.83	.61	.21	.10	.04
13		7.17	2.11	.70	.25	.12	.04
14		8.13	2.39	.80	.40	.13	.05
15		9.08	2.68	.89	.45	.15	.06
16		10.35	3.06	1.01	.51	.17	.07
17		11.62	3.44	1.14	.58	.19	.07
18		12.89	3.82	1.26	.64	.22	.08
19		14.16	4.20	1.39	.71	.24	.09
20		15.46	4.57	1.51	.77	.26	.10
22			5.04	1.76	.93	.31	.12
24			5.95	2.07	1.08	.36	.14
26			7.45	2.46	1.24	.42	.16
28			8.55	2.83	1.42	.48	.19
30			9.67	3.20	1.62	.54	.22
32				3.62	1.80	.59	.25
34				4.04	2.02	.66	.27
36				4.49	2.28	.76	.30
38				4.97	2.52	.84	.33
40				5.45	2.76	.92	.37
42				5.98	3.00	.99	.41
44				6.51	3.27	1.08	.44
46					3.58	1.19	.48
48					3.87	1.29	.52
50					4.17	1.39	.56
52						1.48	.61
54						1.59	.66
56						1.70	.70
58						1.81	.75
60						1.95	.80

Note: Valves below dotted lines are at velocities over 6 feet per second and should be selected with caution

4. When sizing main lines, the size should be determined based on circuits farthest from the source and with the highest operating pressure. (In the example, Section H was sized for requirements of circuit 2).

5. Control valves may be used to reduce the pressure to the lateral lines when balancing the system.

6. Lead-in lines (such as Section G of the previous example) may also be sized to reduce pressures, with the maximum velocity of 6 ft./sec.

4.46 PROVIDE MISCELLANEOUS SYSTEMS

Once all pipes have been sized, it is necessary to determine the design water and pressure requirements, provide for drainage of the lines, and provide sanitary safeguards for cross connections if the source is also for domestic uses.

Drainage - A complete drainage system must be provided for all sprinkler system lines to provide protection from damage due to freezing. To insure complete drainage of the system, two methods for draining the system should be provided. A series of quick coupler valves should be placed within the system to allow the lines to be blown out with an air compressor. Since it is nearly impossible to blow all of the water out of the lines, manual drain valves should be placed at each low point within the system. These will allow any water collecting in low points to be discharged. Most competent landscape contractors are familiar with the procedures required to blow out sprinkler lines. Since this is a complicated procedure, a provision should be included in the special provisions requiring the contractor to demonstrate the procedures to the Department of Highways Maintenance Personnel at the end of the first irrigation season.

Water and Pressure Requirements - Since most water supplies for our landscaping projects are from wells, only those systems utilizing a pump will be discussed here. These systems will be for either an interchange or rest area landscaping project.

In an interchange landscaping project, the only water and pressure requirements are those for the sprinkler system. The capacity and discharge pressure specified for the pump must be equal to the greatest water demand for any circuit and the highest operating pressure plus losses for any circuit, respectively. These losses shall include control valve loss, friction loss, elevation loss or gain, plus 2 psi for miscellaneous losses.

In a rest area landscaping project, the water pressure requirements are generally established by the comfort station fixtures and the water demand by the sprinkler system. Therefore, the capacity specified for the pump must be equal to the greatest water demand for any circuit and the pressure as discussed in Section 4.3, Water Supply. If the sprinkler system is to be used in conjunction with a pressure cell in the comfort station, some means of stablizing the pressure to the sprinkler system may be necessary. This can be done by overriding the pressure switch with the pump control switch on the automatic controller or by using a pressure reducing valve. Caution should be used when overriding the pressure switch with the pump control switch to see that the maximum working pressure of the pressure cells is not exceeded.

Sanitary Safeguards - When the source of water for the sprinkler system is also used for drinking water or other domestic consumption, the Department of Health and Environmental Sciences requires that some form of protection be provided so contaminated water from the sprinkler lines cannot get back to the source. There are several devices available which prevent backflow of contaminated water. The two most suited for rest area use are the pressure vacuum breaker and the reduced pressure backflow preventor. The reduced pressure backflow preventor is more expensive but provides better protection and can be installed anywhere. The pressure vacuum breaker cannot be subjected to back pressures and, therefore, must be installed at least twelve (12) inches above the highest sprinkler head on the system.

4.47 PIPE SIZING COMPUTER PROGRAM

Name: SPR   Language: Basic   Input Format: Data (910)

Purpose: Sizes pipe for sprinkler systems using the friction factor method.

Required Input Data: The flow in gallons/minute and the length of each section of pipe are read in with data statement 910 (and 911, 912, etc. if needed). These valves are read in coordinates (flow, length, flow, length,.....) starting with the Section farthest from the valve and working towards the valve. The program is then "run". The class of PVC pipe to be used (160 or 200) is input along with the allowable friction loss between the last head and valve and the number of sections.

Abstract: This program sizes pipe by the friction factor method using the Hazen - Williams formula. A friction factor is computed using the allowable loss input. The smallest size of pipe whose friction factor is below the allowable friction factor and whose velocity is below 6 ft./sec. is chosen for each section. The flow, size of pipe, length, velocity, friction factor, pressure loss, and cumulative pressure loss are printed for each section.

Limitations: This program will work only for Class 160 or 200 PVC pipe. The maximum number of sections that the computer will handle is 25. The program will not use pipe larger than 2 1/2 inch and prints an error message if larger pipe is required.

Example Run

```
      edit spr basic
EDIT
      circuit no. 1
EDIT
      910 data 3,15,6,15,9,15,12,15,15,15,18,8,36,120
EDIT  run
PRESSURE LOSSES FOR PVC PIPE

INPUT CLASS OF PIPE (160 OR 200)
  ?   200
 INPUT ALLOWABLE PRESSURE LOSS AMD NO. OF LINES
  ?   9,7


LINE  FLOW  SIZE  LENGTH  VEL.  F.F.  HD LOSS  TOTAL HD LOSS
 1     3.0  0.50    15    2.4   1.65    0.25      0.25
 2     6.0  0.75    15    2.8   1.68    0.25      0.50
 3     9.0  0.75    15    4.2   3.55    0.53      1.03
 4    12.0  1.00    15    3.5   1.82    0.27      1.30
 5    15.0  1.00    15    4.3   2.74    0.41      1.72
 6    18.0  1.00     8    5.2   3.84    0.31      2.02
 7    36.0  1.50   120    5.0   2.30    2.76      4.79
VALVE HEAD LOSS
ELEVATION HEAD LOSS
EDIT
```

REFERENCES

Circular No. 18, Approved Arrangement of Valves for Existing Cross Connections, Montana State Board of Health, Helena, Montana.

Residential Sprinkler Design Guide, Rain Bird Sprinkler MFG. Corporation, Glendora, California, 1972.

SECTION 4.5
WASTE DISPOSAL

4.5 WASTE WATER DISPOSAL

Introduction

Because of more stringent requirements and awareness about ecology, the task of providing adequate sewage treatment at rest areas has become more and more difficult. At present there are many different methods of sewage treatment; however, the wide fluctuations in use and expected growth in use present a major problem in design. Because of this, such conventional methods as trickling filters, contact stabilization, and conventional activated sludge are usually not considered for rest areas. The advantages, disadvantages, and general design criteria for two types of sewage treatment, septic tank - drainfield systems and stabilization ponds, will be discussed here. Package plants such as the oxidation ditch and ultra filtration which are discussed briefly should not be ruled out; however, they should be considered somewhat experimental.

Each method of sewage treatment has its particular advantages and disadvantages. The designer should consider the following factors before making a selection: site topography, availability of water source for dilution of sewage effluent, rest area site development (particularly water supply), adjacent development, soil conditions, climate, and cost.

Before individual methods of sewage treatment can be discussed, the constituents of sewage must be discussed. Sewage is 99.9% water by weight; however, it is the 0.1% solids that cause the pollution problems. The total amount of solids consists of inorganic and organic solids. Inorganic solids are those solids which are inert and are not subject to decay, while organic solids are those solids which are subject to decay and decomposition through the activity of bacteria and other living organisms. This decay or decomposition can be accomplished by anaerobic or aerobic decomposition. Anaerobic decomposition is accomplished by anaerobic bacteria without dissolved oxygen while aerobic decomposition is accomplished by aerobic bacteria in the presence of dissolved

oxygen. Dissolved oxygen is the free or molecular oxygen present in water or sewage, and the BOD (biochemical oxygen demand) is the amount of oxygen required for aerobic decomposition of the unstable organic material in sewage. BOD should be considered as negative dissolved oxygen. The primary purpose of sewage treatment is to neturalize this BOD demand and return the sewage effluent to the environment in a condition that it can support aquatic life.

4.51 WASTE LOADING

Regardless of the method of sewage treatment, the design organic and hydraulic loadings must be determined. These design loadings are dependent upon the number of rest area users which can be determined from the following equation:

$$N = ADT \times DF \times S \times P \times SPK$$

where:

N = Number of rest area users on the design day

ADT = Estimated Average Daily Traffic for the design year

DF = Direction Factor (1.0 if both directions served, 0.5 if only one direction is served)

S = Percent of vehicles stopping at the rest area per day (between 5 and 14 percent of the passing traffic will stop with the higher percentage occurring at the more remote locations)

P = Number of people per vehicle using comfort facilities (usually 2.25)

SPK = Seasonal peak factor (usually 1.30)

Once the number of users has been determined, the loadings can easily be determined from the following equations:

Organic Loading

Lbs. of BOD = N x .02 lbs. of BOD per user

Hydraulic Loading or design flow

Q = N x 5 gals. per user

4.52 SEPTIC TANK - DRAINFIELD SYSTEM

At many rest areas, the sewage can be disposed of by a septic tank system. A septic tank system is an entirely underground method of sewage disposal consisting of a septic tank and an absorption field (drain tile field). Since the sewage is disposed of underground, a visible effluent is not produced, which is one of the primary advantages of a septic tank system. A typical septic tank system serving a rest area is shown in figure 4.36.

Sewage from the rest area discharges into the septic tank. Sedimentation takes place in the upper portion of the tank and the accumulated sludge undergoes anaerobic decomposition in the bottom of the tank. The heavier sewage solids settle to the bottom of the tank, forming a blanket of sludge. The lighter solids including fats and greases, rise to the surface and form a layer of scum. A considerable portion of the sludge and scum are liquified through decomposition or digestion. The sewage is discharged to the absorption field by a dosing siphon which floods the drain tile pipes and surrounding gravel filter. The sewage then percolates into the soil or is drawn up by the root system of the vegetation and evapotranspired into the atmosphere. The absorption field does not provide treatment of the effluent, merely disposal.

In the preliminary design stage, the feasibility of disposing of the sewage by a septic tank system should be determined. In order to use a septic tank system, the soil must meet the following conditions:

1. The percolation rate must not exceed 60 minutes per inch (or not less than one inch per hour).

2. The maximum seasonal elevation of the groundwater should be at least 4 feet below the bottom of the absorption trench or seepage pit.

3. Rock or other impervious strata must be at least 4 feet below the bottom of the trench or seepage pit.

Since only approximately 35% of the suspended solids are removed in the

septic tank, a steady flow of solids is discharged to the drain tile field. Over the years these particles will clog the drain tile field requiring its replacement. For this reason, many engineers consider the septic system as a temporary installation.

The purpose of the septic tank is to extend the life of the drain tile field as long as possible. The septic tank should remove as many of the solids in the incoming sewage as possible before discharging it into the absorption field. By dividing the tank into several compartments, the amount of suspended solids removed can be increased. For practical purposes, the tank is usually only divided into two compartments. Each compartment should be provided with an inspection manhole so that the amount of scum and solids present in the tank can be observed. The inlet and outlet to each compartment should be made with tees or baffles to retain as many of the solids as possible. The tank should be structurally sound, watertight, and constructed of reinforced concrete.

The septic tank should provide a detention period of 24 hours and an additional volume of storage should be provided for the accumulation of sludge. This volume should equal approximately 25% of the design flow. The capacity of the tank can be determined by the following equation:

$$V = 1.25\ Q$$

where:

V = Net capacity of the tank - gals.

Q = Hydraulic Loading - gals. per day

The minimum capacity of the septic tank should be at least 1,000 gallons and have a minimum depth of 4 feet. A well proportioned tank is obtained by letting the width equal the depth and making the length 2 to 3 times the width. The tank should be divided into two compartments with the length of the first compartment equaling 2/3 of the total length required.

A dosing tank should be used in conjunction with the septic tank in order to

SEPTIC TANK SEWAGE DISPOSAL SYSTEM

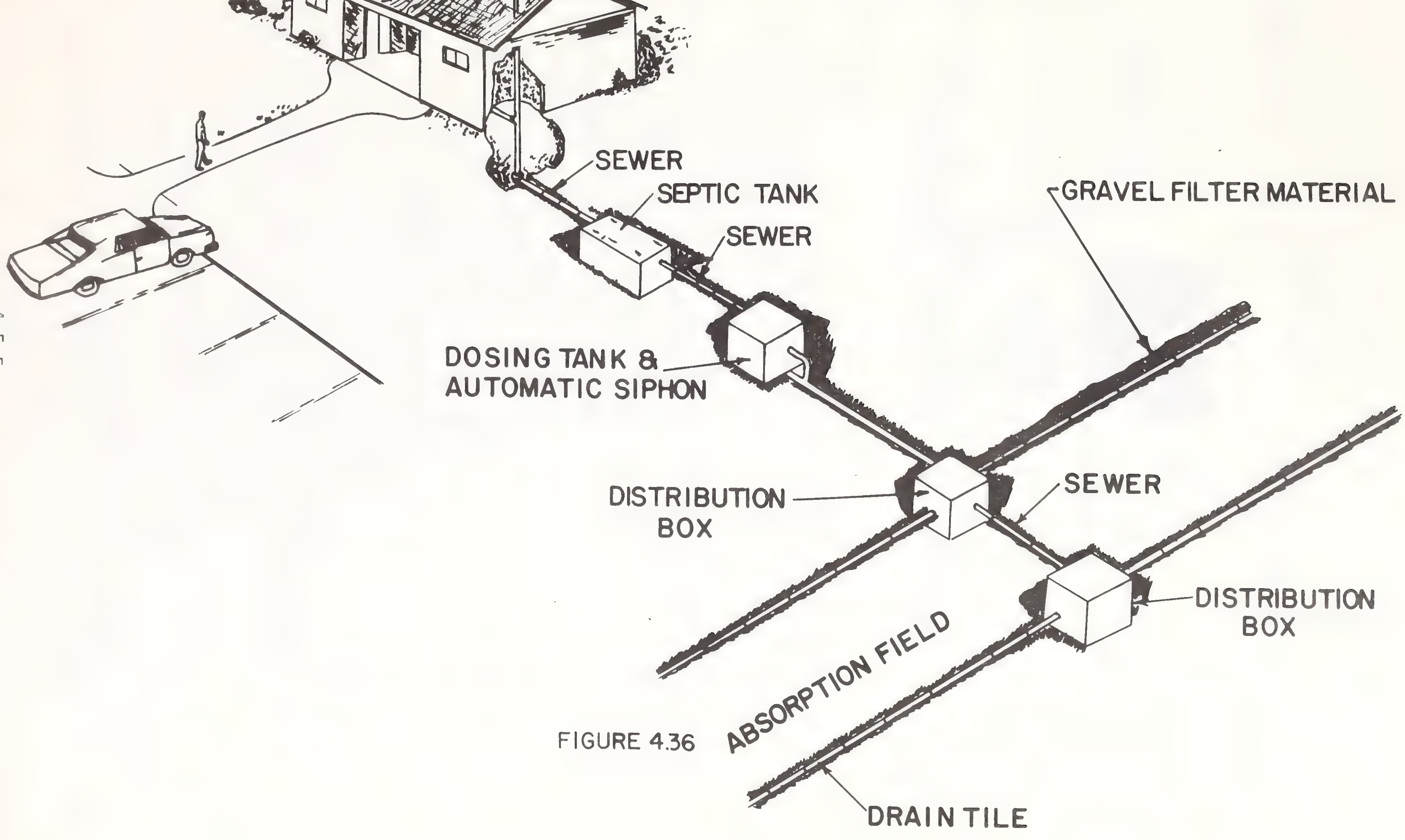
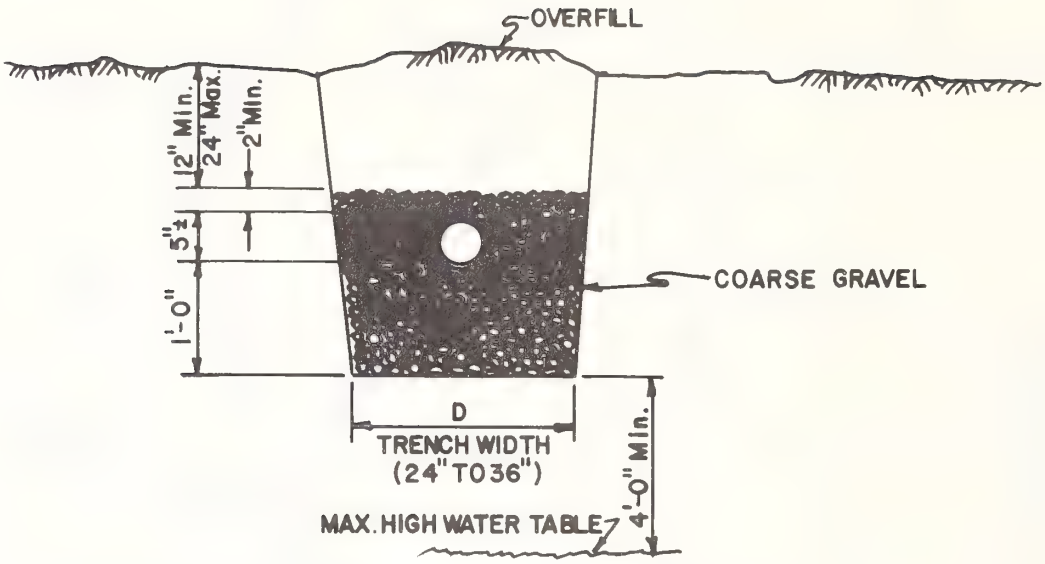
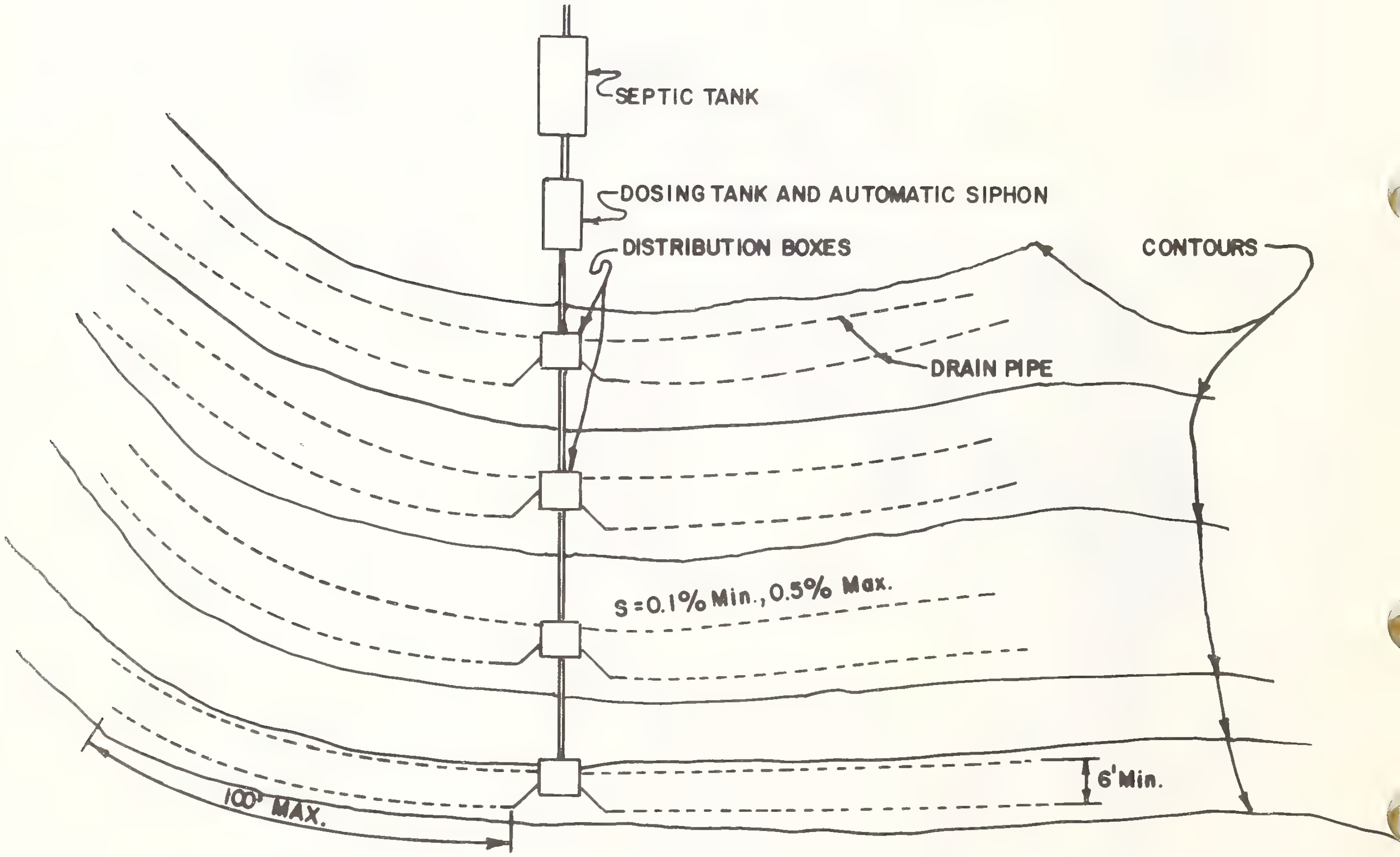


FIGURE 4.36



TYPICAL TRENCH SECTION



TYPICAL DRAIN FIELD

FIGURE 4.37

obtain proper distribution of sewage throughout the disposal area and give the absorption bed a chance to rest or dry out between dosings. The tank should have an effective capacity equal to about 75 percent of the interior capacity of the drain pipe to be dosed at one time and should be provided with an automatic siphon to discharge the tank.

When the total length of the drainfield trench exceeds 1000 feet, the dosing tank should be provided with two siphons dosing alternately and each serving one-half of the drainfield.

Dosing siphons require operating heads ranging from about 5 feet down to 1 or 2 feet. Care should be taken to specify a siphon and to design a system that is compatable with this head loss.

The absorption system should be a drainfield. The drainfield provides distribution of the sewage and allows evapotranspiration.

The drainfield is a shallow trench type of absorption system consisting of a series of trenches containing drain tile or perforated pipe laid in gravel. A typical layout of a drainfield and a typical trench are shown in Figure 4.37. The trenches should be of equal length; however, they should not exceed 100 feet. The width of the trench can vary between 2 feet and 3 feet, and a minimum spacing of 6 feet between trenches should be provided to prevent short-circuiting. These requirements are necessary to insure uniform distribution of the sewage to the field. The slope of the drain tile should not be less than 2 inches per 100 feet or exceed 6 inches per 100 feet, and the pipe shall be placed parallel to a contour. The trench should have a minimum cover of 12 inches of earth, and the depth of the trench should not exceed 3 feet.

If a suitable location for a drainfield is not readily available, it may be possible to use a sewage lift station to pump the sewage to a more suitable location. It is recommended that package lift stations be used in these situations.

The total length of the dispersal lines that make up the drainfield depends upon the hydraulic loading, the soils ability to absorb sewage, the trench width and the depth of gravel beneath the drain pipe. The hydraulic loading is discussed in Section 4.51. The soils ability to absorb is measured by its percolation rate as determined from field tests.

The rate of which sewage may be applied to a soil absorption system can be determined from Figure 4.38 using the percolation rate. The daily design flow is divided by the allowable rate of sewage loading (gals/sq. ft./day) to obtain the required square feet of trench. This area is then divided by the width of the trench to obtain the linear feet of trench required. Figure 4.38 is based on a gravel depth of one foot below the drain pipe.

In situations where land available for the drainfield is minimal, it is possible to reduce the length of trench required by increasing the trench width or the depth of gravel below the drain tile. The Manual of Septic Tank Practice details the procedure for doing this.

The percolation rate determined from preliminary percolation tests should be used to design the drainfield and estimate the total length of trench required. However, the contractor should be required to make percolation tests at the time he is constructing the drainfield and should use the percolation rate determined from those tests to establish the length of trench. This requires that a table giving the required trench length for various percolation rates be included in the plans. The following example shows the development of such a table:

Example: Given: ADT = 6000

Assume: S = 12%, P = 2.25, SPK = 1.30

Number of Users = 6000 x .5 x .12 x 2.25 x 130 = 1053

Hydraulic Loading = 1053 x 5 = 5265 gal/day

For t = 1 min, from Figure 4.38, Q = 5 gal/sq. ft./day, $^{5265}/_{5}$ = 1053 sq.ft.

for 2 foot wide trench $L = {}^{1053}/_{2} = 526$ ft.

For t = 2 min from Figure 4.38, Q = 3.5 gal/sq.ft./day, $^{5265}/_{3.5}$ = 1500 sq.ft.

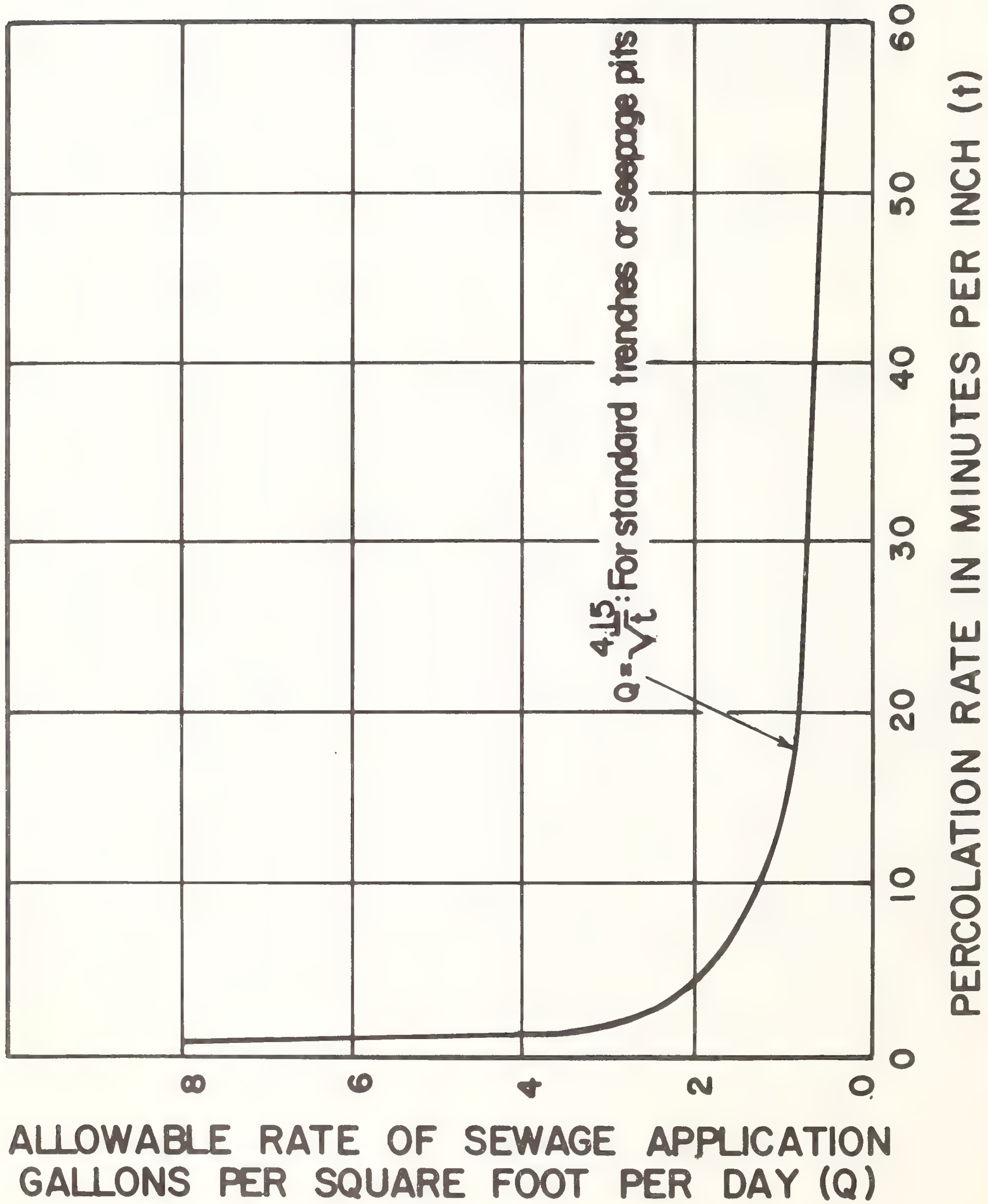
for 2 foot wide trench $L = \frac{1500}{2} = 750$ ft.

MINIMUM REQUIRED ABSORPTION SYSTEM(1)

Percolation Rate (time in minutes for water to fall 1")	D(3) (drainfield trench width in feet)	Drainfield Min. Required Length in Feet
1 or less	2	526
2	2	750
3	2	912
4	2	1053
5	2	1177
10	3	1110
15	3	1359
30	3	1923
45	3	2355
60(2)	3	2719

(1) For ADT = 6000
(2) Percolation rate over 60 minutes unacceptable for drainfield
(3) Wider trenches can be used where space is limited

Because the septic tank - drainfield system does not treat the sewage but merely disposes of it, it is necessary to see that minimum distances between the system and water sources are maintained to prevent contamination. Underground contamination may travel in any direction and for considerable distances unless effectively filtered by the soil. Normally, the contamination moves in the same direction as the flow of the ground water, from areas of high water table to areas of lower water tables. In general, the water table follows the contour of the ground surface. For this reason, disposal systems should be located downhill from wells or springs. Minimum distances to be maintained for the septic tank and drainfield are shown in Table 4.30.



$Q = \frac{4.15}{\sqrt{t}}$: For standard trenches or seepage pits
PERCOLATION RATE IN MINUTES PER INCH (t)
0
10
20
30
40
50
60
ALLOWABLE RATE OF SEWAGE APPLICATION
GALLONS PER SQUARE FOOT PER DAY (Q)
0
2
4
6
8

FIGURE 4.38

TABLE 4.30

Minimum Distances in Feet

From	Septic Tank	Nearest Point in Drainfield
Well	50	100
Foundation Wall	5	5
Water Lines	10	10
Stream or Lake	50	100

4.53 STABILIZATION PONDS (LAGOONS)

Raw sewage can be successfully treated by storing in a large shallow basin. Here climatic and biological factors combine to treat the sewage. The effluent from the treatment basin can then be discharged into an evaporation pond where the surface area is large enough to allow for total evaporation of the effluent.

The treatment basin is called a stabilization pond or lagoon. The stabilization pond process depends upon aerobic bacteria to decompose (and stabilize) the organic solids in the incoming sewage. Because the process is aerobic (versus anaerobic in the septic system) no obnoxious odors are produced. The dissolved oxygen necessary for aerobic conditions is obtained in two ways. First, because of the large surface area and shallow depth a great deal of oxygen is absorbed from the atmosphere. The wind aids in this absorption by creating turbulence on the pond surface. The other source of oxygen is algae. Algae uses the nutrients in the sewage, dissolved carbon dioxide, and sunlight and the photosynthesis process to produce oxygen. The dissolved oxygen satisfies the biochemical oxygen demand (BOD) of the sewage. Figure 4.39 shows these interacting processes.

The evaporation pond is simply a pond which is sized to allow for total evaporation of the sewage effluent. Since there is no effluent discharged, surface water pollution is impossible.

<u>Treatment Basin Design</u> - The treatment basin or primary cell should be circular or nearly so in order that solids do not drift to sheltered corners. The raw sewage should be introduced near the center of the pond. Recommended side slopes are 4:1. The minimum operating liquid depth is 2 feet and the maximum liquid depth should be 5 feet with 2 feet of free board. The cell should be provided with a positive seal so that adequate water levels can be established and maintained. Figure 4.40 shows recommended dimensions and details for the

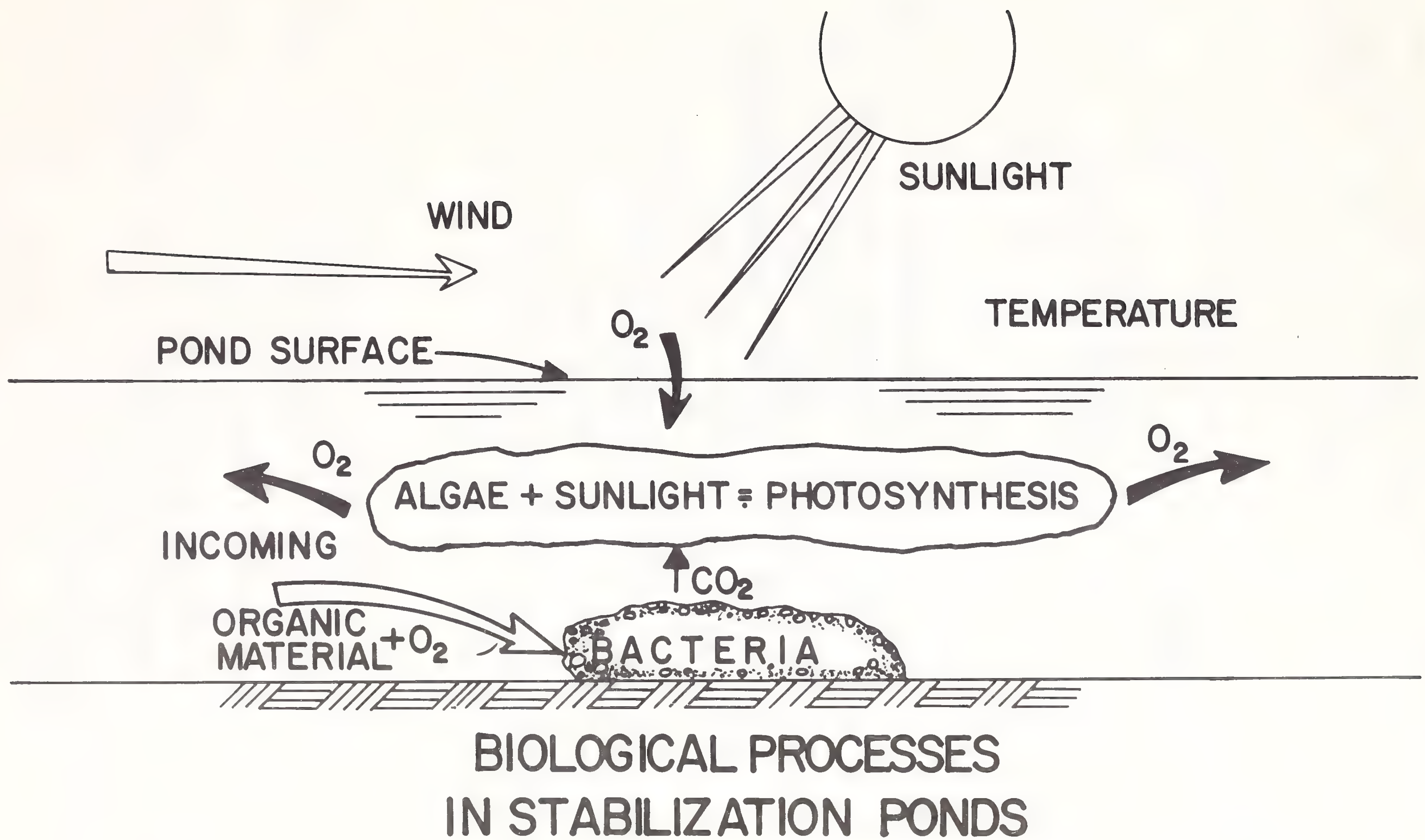

SUNLIGHT
WIND
TEMPERATURE
O_2
POND SURFACE
O_2
ALGAE + SUNLIGHT = PHOTOSYNTHESIS
O_2
INCOMING
CO_2
ORGANIC
MATERIAL $+O_2$
BACTERIA
BIOLOGICAL PROCESSES
IN STABILIZATION PONDS

Figure 4.39

treatment cell.

The size of the primary cell is determined by the organic and hydraulic loading. The surface area must be great enough to insure that enough oxygen is absorbed to oxyidize the BOD and keep the process aerobic. The allowable rate of organic loading varies from 20 to 50 pounds per acre of surface area per day. The 50 pounds per acre per day is recommended for design. The required surface area is determined by the following equation:

$$\text{Required surface area} = \frac{\text{Organic Loading}}{\text{Allowable Pond Loading}}$$

This required surface area is for an operating depth of 5 feet.

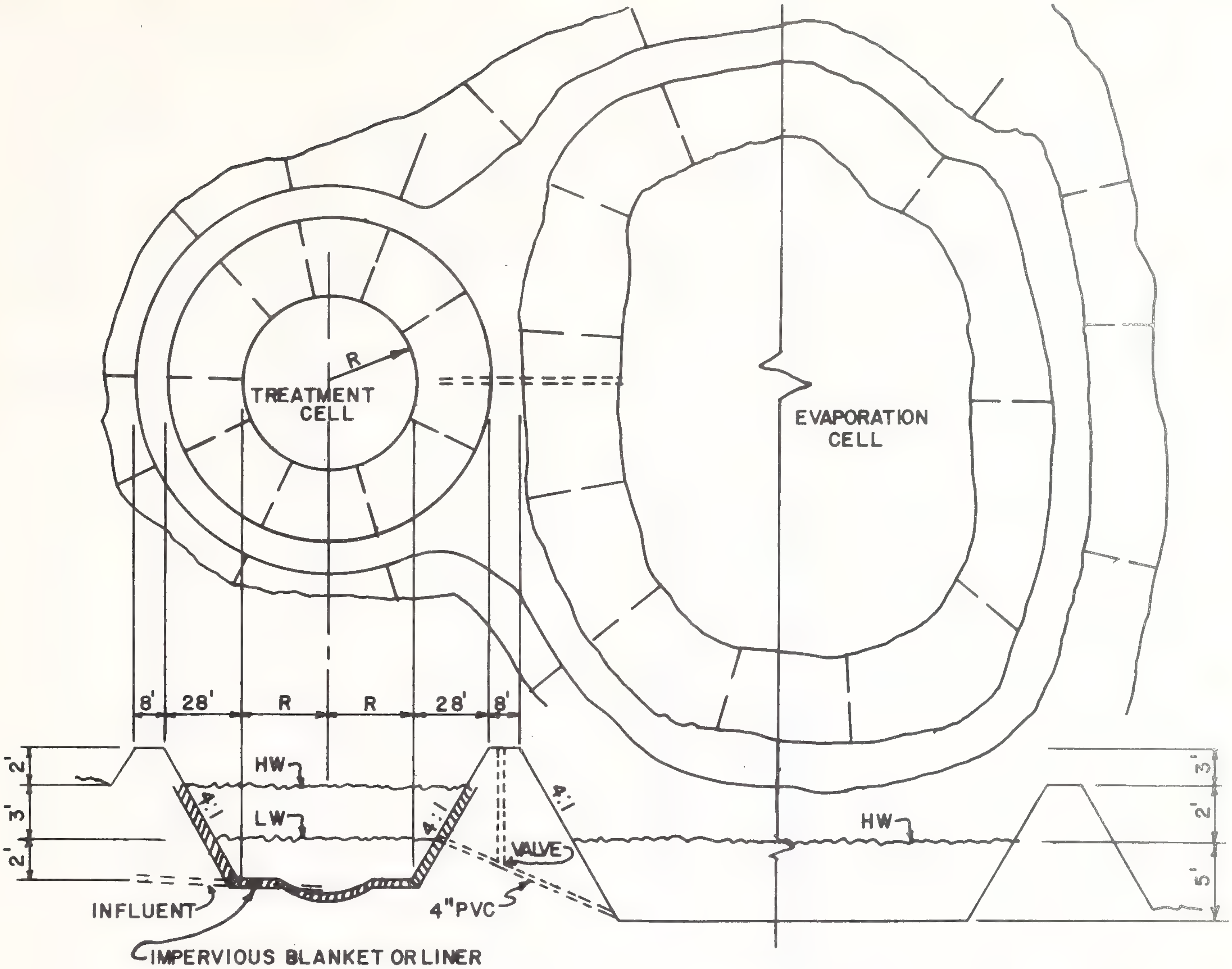
The volume of the primary cell must be great enough to provide a 7 to 30 day detention time. This detention time is required to give the bacteria time to fully oxydize the BOD. A 20 day detention time is recommended for design. The required volume is determined by the following equation:

$$\text{Required pond volume} = \text{Hydraulic loading} \times \text{Required detention time}$$

The required volume is also at a pond depth of 5 feet.

With the required surface area and volume known, Table 4.31 can be used to select the radius of the pond. The pond with the smallest radius but which still provides the required surface area and volume should be used. The areas and volumes in Table 4.31 are for a round pond with 4:1 side slopes and a depth of 5 feet.

Whenever the required radius of the treatment pond becomes excessive, over 30 or 35 feet, serious consideration should be given to using two smaller cells operated in parallel. This will permit the use of only one cell during low-flow periods and both could be put into service during the higher loading periods. It would also permit one cell to be taken out of service for maintenance without disturbing the operation.



TYPICAL STABILIZATION POND
Figure 4.40

TABLE 4.31

Volume and Surface Area for a Round Pond
(depth = 5', 4:1 Side Slopes)

Radius (ft)	Volume (ft^3)	Area (ft^2)	Radius (ft)	Volume (ft^3)	Area (ft^2)
1	3471	1385	24	19729	6082
2	3833	5121	25	20813	6362
3	4225	1662	26	21928	6648
4	4650	1810	27	23075	6940
5	5105	1963	28	24253	7238
6	5592	2124	29	25463	7543
7	6110	2290	30	26704	7854
8	6660	2463	31	27976	8171
9	7241	2642	32	29280	8495
10	7854	2827	33	30615	8825
11	8498	3019	34	31981	9161
12	9173	3217	35	33379	9503
13	9880	3421	36	34809	9852
14	10619	3632	37	36270	10207
15	11388	3848	38	37762	10568
16	12189	4072	39	39286	10936
17	13022	4301	40	40841	11310
18	13886	4536	41	42427	11690
19	14781	4778	42	44045	12076
20	15708	5027	43	45695	12469
21	16666	5281	44	47375	12868
22	17656	5542	45	49087	13273
23	18677	5809			

Evaporation Pond Design - The shape of the evaporation disposal pond is not critical so the pond can be placed and shaped to fit available right-of-way. The evaporation pond also serves as a secondary treatment basin so the maximum operating depth should be 5 feet to insure that the process stays aerobic. The side slopes should be 4:1 with 2 feet of freeboard above the 5 foot operating depth. A seal should be provided to prevent groundwater contamination although the seal is not as important as for the treatment basin. Bentonite or clay should suffice.

The required surface area of the evaporation pond is determined using net evaporation rate. The net evaporation is equal to the average annual evaporation minus the average annual precipitation. The average annual evaporation can

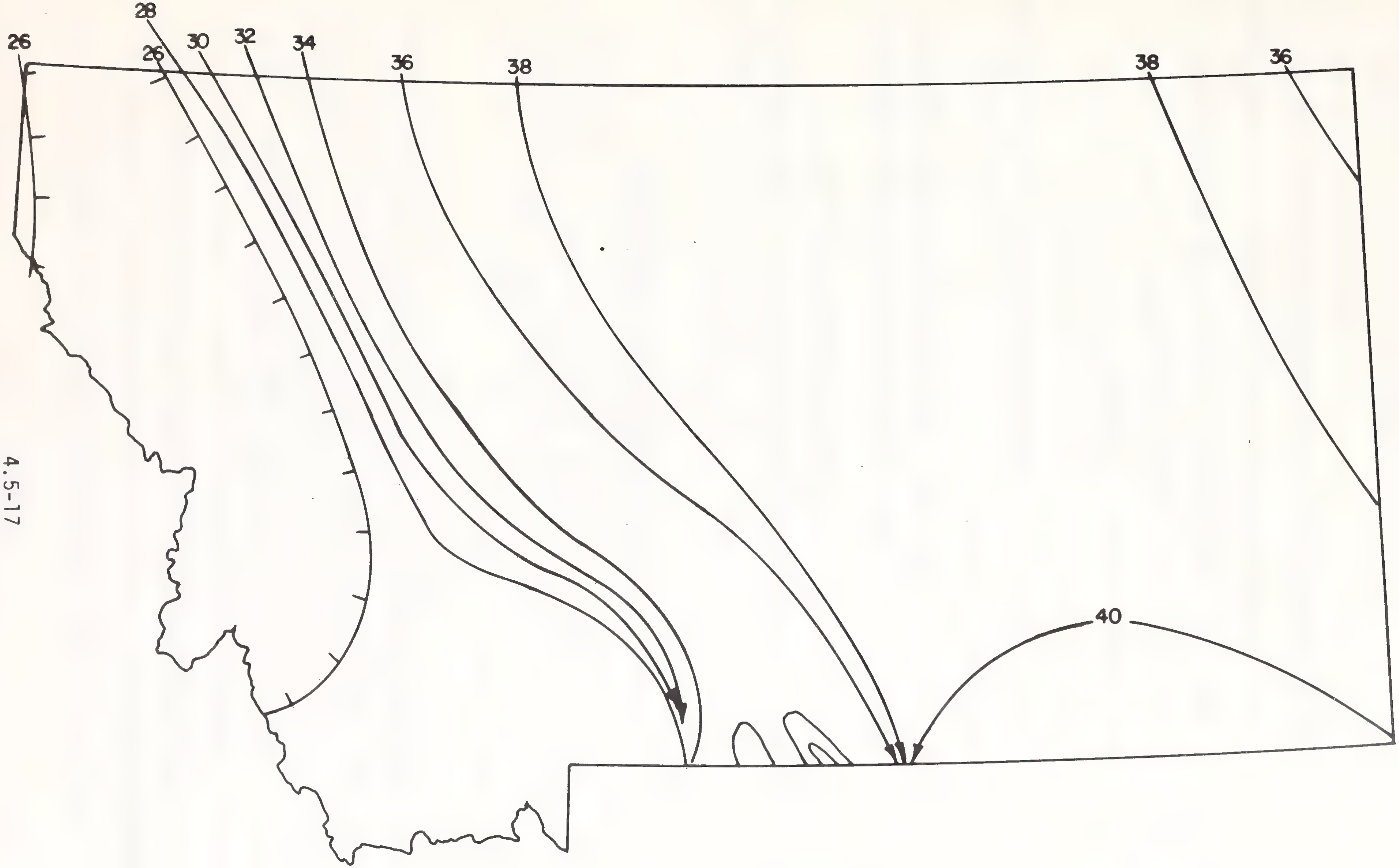


FIGURE 4.41 - AVERAGE ANNUAL EVAPORATION (INCHES) FROM SHALLOW LAKES.

be determined from Figure 4.41. The average annual precipitation can be determined from Figure 3.2.

Because of the great fluctuations in both evaporation and rest area use with the various seasons, evaporation ponds should be designed using annual flows rather than daily flows. The annual hydraulic loading can be computed by:

$$Q_A = ADT \times DF \times S \times P \times 5 \text{ gal/user} \times 365 \text{ days}$$

where Q_A = annual hydraulic loading in gallons

ADT = estimated average daily traffic for the design year

DF = direction factor (1.0 if both directions served and 0.5 if only one direction is served)

S = Percent of vehicles stopping at the rest area per day (between 5 and 14 percent of the passing traffic will stop with the higher percentage occurring at the more remote locations)

P = Number of people per vehicle using comfort facilities (usually 2.25)

The surface area required to dissipate the annual hydraulic loading by evaporation can be calculated by:

$$\text{Required surface area} = \frac{\text{annual hydraulic loading in gallons}}{7.48 \times \text{net annual evaporation in feet}}$$

The surface area of the treatment basin can be deducted from the required surface area to determine the surface area of the evaporation pond. This is the surface area at an operating depth of 5 feet.

Stabilization Pond Operation - The design of a stabilization pond requires that the designer have a good knowledge of the operational procedures and maintenance of the pond. Stabilization ponds require a minimum of attention from the operator; however, failure to follow the simple operating procedures which are required can severely affect the pond's efficiency. Operators should plan to visit the facility at least twice each week to perform routine operational and maintenance tasks.

When the new pond is first put into operation, a satisfactory water depth should be obtained in the shortest possible time. This helps retard unwanted weed growth. When filling a two-cell unit, all waste water should be routed into one-cell until the desired operating depth of 5 feet is reached. The interconnecting structure can then be opened and both cells operated at a uniform depth. The flow should then be divided between the two-cells until the operating depth is reached in each cell. The pond overflow can then be allowed to discharge into the evaporation pond.

For most efficient operation of the pond, the depth of water should be adjusted according to the season. All depths of water referred to are in the treatment cell or cells. Any excess water that must be discharged when adjusting the water level should simply be discharged into the evaporation basin.

During the summer the depth of the water in the pond should be raised to full operational level (5') and in two cell units the depths should be equalized.

During the late fall (late October - early November) the water level in the pond should be reduced to about 2 to 3 feet in depth. This will allow ample storage space for the winter months when the pond is again operated on a full retention basis. If flows are low enough one cell may be taken out of operation if it is a two cell pond.

During the winter if traffic at the rest area is low and discharge from the rest rooms into the lagoon is less than normal, the attendant should allow extra water to flow into the sewer system to maintain the operating depth in the pond. Once the pond is frozen over, the water level should be dropped several inches to bring air in contact with the water.

If the pond is up to maximum operational level at the time of spring break up it should be drawn to or near minimum operational level in anticipation of the summer loading. Odor problems may exist for approximately a week after the

ice goes off as the pond changes from an anaerobic process to aerobic. If the odor persists and becomes a nuisance, the pond can be mechanically aerated or sodium nitrate can be added at the rate of 100 pounds per acre per day until the odor disappears.

4.54 PACKAGE PLANTS

"Package Plants" can be used to provide suitable sewage treatment at rest areas. A package plant is a complete sewage treatment plant usually manufactured by one company for rather small sewage treatment applications. The design of the package plant is left to the manufacturer, but he must be given the design loadings and user characteristics. Package plants operate on one or a combination of several basic sewage treatment processes and the names of some manufacturers who build plants which use these processes follow.

Extended Aeration - This sewage treatment process involves the decomposition of liquid sewage by biological oxidation and the reduction of sewage solids by aerobic digestion. The latter function is achieved by continuous aeration resulting in the addition of oxygen to the sewage until an acceptable percentage of solids has been converted to inert sludge. Turbulence in the aeration tank aids the digestion process by rapidly mixing the fresh sewage solids with the activated sludge, by breaking up the sewage solids and by bringing the contents of aeration tank in contact with the atmosphere where additional oxygen may be obsorbed.

Smith and Loveless, Marolf Hygienic Equipment, Inc., Fram Corporation, and Can Tex Industries all manufacture extended aeration treatment plants.

Oxidation Ditch - The oxidation ditch process is a modified form of the activated sludge process, and may be classified in the complete mix, long-term aeration group. The system consist of an elongated oval ditch which forms a complete circuit, a Cage Rotor and a final settling tank.

Raw sewage passes directly into the ditch where it is mixed with the active organisms. The Cage Rotor serves as an aeration device that entrains the necessary oxygen into the liquid and keeps the contents of the ditch mixed and moving. Mixed liquor in the ditch flows to the settling tanks for separation. The

clarified liquid then passes over the effluent weir through the chlorination tanks and to the receiving stream. All settled sludge and floating scum are removed from the settling tank and returned to the ditch.

Physical and biological processes are carried out in the oxidation ditch. A small portion of the organic matter undergoes direct chemical oxidation, but the bulk of the organic matter must be stabilized by the biochemical activities of the micro-organisms previously formed in the system.

The oxidation ditch is not a true package plant and the components must be designed and specified as explained in the Water Supply and Waste Disposal Series, Volume 6, "Oxidation Ditch Sewage Waste Treatment Process", published in April, 1972, by the Federal Highway Administration.

Ultra Filtration - The ultra filtration process is a closed system which is based upon the separation of wastes from the transport medium. A fluid medium (usually a type of oil) collects the sewage and transports it to a separation tank where the wastes are separated from the medium. The medium is then filtered, treated and recycled to the sanitary fixtures. The sewage is routed to a holding tank where it is incenerated and turned to ash.

Chrysler Corporation and Thiokol Chemical Corporation are two known manufacturers of ultra filtration processing plants.

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SECTION 4.6
OPEN CHANNEL FLOW

4.6 OPEN-CHANNEL FLOW

Introduction

The flow of water in a conduit may be either open-channel flow or pressure flow. The two kinds of flow are similar in many ways but differ in one important respect. Open-channel flow must have a free surface subject to atmospheric pressure, wheras pressure flow has none, since the water must fill the whole conduit. Open-channel flow includes the flow in irrigation ditches, rivers, streams, roadside ditches, and usually storm sewers.

This section will discuss the basic hydraulics of open-channel flow, including Manning's Equation, and present charts for the solution of Manning's Equation for the more common channel sections. Computer programs which aid in the solution of open-channel flow are also presented.

The charts and much of the information presented in this section are taken from two publications by the Federal Highway Administration entitled Hydraulic Design Series No. 3, "Design Charts for Open-Channel Flow", and Hydraulic Design Series No. 4, "Design of Roadside Drainage Channels".

4.61 PRINCIPLES OF FLOW IN OPEN-CHANNELS

Types of Flow - Flow in open channels is classified as steady or unsteady. The flow is said to be steady when the rate of discharge is not varying with time. In this section, the flow will be assumed to be steady at the discharge rate for which the channel is to be designed. Steady flow is further classified as uniform if velocity and depth of flow are constant, and nonuniform or varied if velocity and depth of flow changes from section to section. Although most drainage channels are designed on the assumption of uniform flow, a knowledge of varied flow is needed to solve the more complex flow problems. Another classification of flow, subcritical (tranquil) or super-critical (rapid or shooting), will be discussed in later paragraphs.

Uniform Flow - To have uniform flow, the grade must be constant and all cross sections of flow must be identical in form, roughness, and area, necessitating a constant mean velocity. Under uniform flow conditions, the depth (d_n) and the mean velocity (V_n) for a particular discharge are said to be normal. Under these conditions, the water surface is parallel to the stream-bed (Fig. 4.41). Normal depth is also defined as the depth at which uniform flow will occur when a given quantity of water flows through a long channel of uniform dimensions, roughness (n), and slope (S_o).

Uniform flow conditions are rarely attained in drainage channels, but the error in assuming uniform flow in a channel of fairly constant slope and cross section is small in comparison to the error in determining the design discharge. If the channel cross section, roughness, and slope are fairly constant over a sufficient distance to establish essentially uniform flow, equation, such as that of Manning give reliable results.

The Manning Equation - Water flows in a sloping drainage channel because of the force of gravity. The flow is resisted by the friction between the water and the wetted surface of the channel. The quantity of water flowing (Q), the

depth of flow (d), and the velocity of flow (V) depend upon the channel shape, roughness, and slope (S_o). Various equations have been devised to express the flow of water in open channels. A useful equation for channel design is that named for Robert Manning, an Irish Engineer. The Manning equation for velocity of flow in open channels is:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (1)$$

Where

V = mean velocity in feet per second (f.p.s.)

n = Manning coefficient of channel roughness

R = hydraulic radius, in feet

S = slope, in feet per foot. When the Manning equation applies, $S = S_o$.

Chart 4.32 provides a nomograph which aids in the solution of Mannings equation.

The value of the Manning coefficient n is determined by experiment. Some n values for various types of channels are given in Table 4.32.

R, the hydraulic radius, is a shape factor that depends only upon the channel dimensions and the depth of the flow. It is computed by the equation:

$$R = \frac{A}{WP} \qquad (2)$$

Where

A = cross-sectional area of the flowing water in square feet taken at right angles to the direction of flow.

WP = wetted perimeter the length, in feet, of the wetted contact between a stream of water and its containing channel, measured in a plane at right angles to the direction of flow.

Another basic equation in hydraulics is:

$$Q = AV \qquad (3)$$

or discharge (Q) is the product of the cross sectional area (A) and the mean velocity (V).

By combining equation (1) and (3), the Manning equation can be used to compute discharge directly or

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \tag{4}$$

In many computations it is convenient to group the properties peculiar to the cross section, in one term, called conveyance (K) or

$$K = \frac{1.49}{n} AR^{2/3} \tag{5}$$

then

$$Q = KS^{1/2} \tag{6}$$

When a channel cross section is irregular in shape such as one with a relatively narrow deep main channel and a wide shallow overbank channel, the cross section must be subdivided and the flow computed separately for the main channel and for the overbank channel. The same procedure is used when different parts of the cross section have different roughness coefficients. In computing the hydraulic radius of the subsections, the water depth common to two adjacent subsections is not counted as wetted perimeter.

Conveyance can be computed and a curve drawn for any channel cross section. The area and hydraulic radius are computed for various assumed depths and the corresponding value of K is computed from equation (5). The values of conveyance are plotted against the depths of flow and a smooth curve connecting the plotted points is the conveyance of each subsection (k_a, K_b, . . . k_n) is computed and the total conveyance of the channel is the sum of the conveyances of the subsections or K $(k_a + k_b \ldots + k_n)$. Discharge can be computed using equation (6), using the conveyance of the subsection.

The concept of channel conveyance is useful when computing the distribution of overbank flood flows in the stream cross section and the distribution through the openings in a proposed stream crossing. The discharge through each opening can be assumed to have the same ratio to the total discharge as the ratio of

conveyance of the opening bears to the total conveyance of the channel.

Discharges computed by the Manning equation do not have the accuracy to which the computation can be carried. Results of the discharge computations are generally rounded off to avoid an inference of great accuracy.

Energy of Flow - Water flowing in an open channel possesses energy of two kinds - potential energy and kinetic energy. The potential energy is due to the position of the water above some datum, and kinetic energy is due to the velocity of the flowing water. In channel problems, energy is conveniently expressed in terms of head. Thus, a column of water 20 feet high has a potential (static) head of 20 feet with respect to the bottom of the column. Flowing water has both potential head and velocity head, the velocity head being equal to

$$\frac{V^2}{2g}$$

Where

V = the mean velocity in feet per second.

g = acceleration of gravity or 32.2 feet per second per second.

A useful hydraulic concept of the energy of the flowing water within one vertical cross section of the channel is that of specific head (also called specific energy)

$$\text{Specific head } (H_o) = d + \frac{V^2}{2g} \qquad (7)$$

If the potential head is related to some datum (Fig. 4.42) at or below the bed of the channel at the outlet, energy can be expressed in terms of total head. If Z is the elevation of the channel bottom, total head at any section is:

$$\text{Total head } (H) = d + \frac{V^2}{2g} + Z \qquad (8)$$

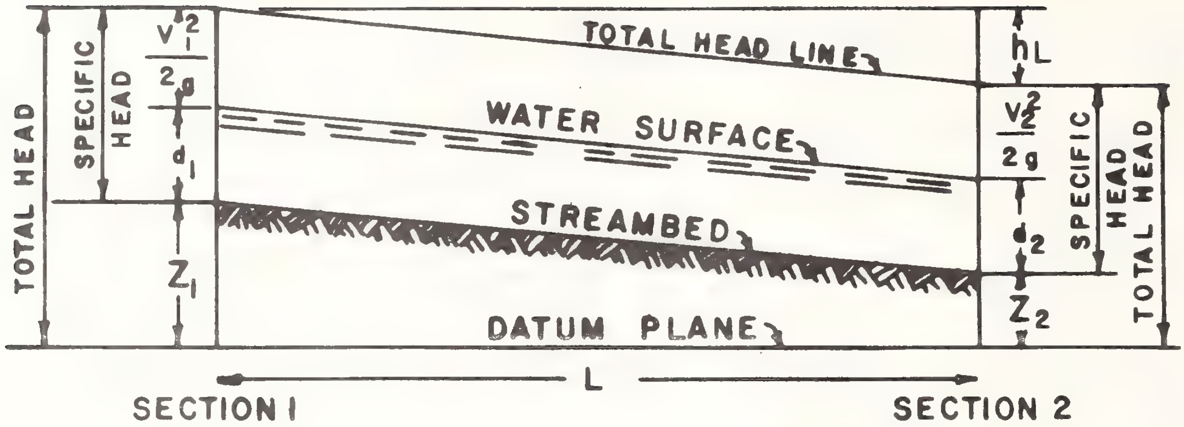


FIGURE 4.42– Water-surface profile of channel with uniform flow.

The energy losses due to friction, channel contractions, changes in alignment, and other factors are termed head losses (h_L). The law of conservation of energy (Bernoulli's theorem) states that the total head at any section is equal to the total head at any section downstream, plus intervening head losses or for the channel in Figure 4.41, is equal to the total head at Section 2, plus head loss between Section 1 and 2, or

$$d_1 + \frac{V_1^2}{2g} + Z_1 = d_2 + \frac{V_2^2}{2g} + Z_2 + h_L \qquad (9)$$

In Figure 4.41, the head loss, in a channel of uniform cross section, equals the change in Z or (Z_1-Z_2). Thus, the water surface is parallel to the streambed, and

$$d_1 + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g}$$

The flow is uniform and can be computed by the Manning equation. The head loss

$$(Z_1 - Z_2) = LS_o \qquad (10)$$

Where

L = horizontal distance between section 1 and section 2

S_o = channel slope or $\frac{Z_1 - Z_2}{L}$

S_o in uniform flow is sometimes called the friction slope. For uniform flow, the Manning equation can be computed for $S(=S_o)$.

$$S = \left(\frac{Vn}{1.49R^{2/3}}\right)^2 \qquad (11)$$

When the head loss does not equal the change in Z, nonuniform flow occurs and the depth of flow either increases or decreases in a uniform channel. In Figure 4.43, flow takes place with decreasing depth.

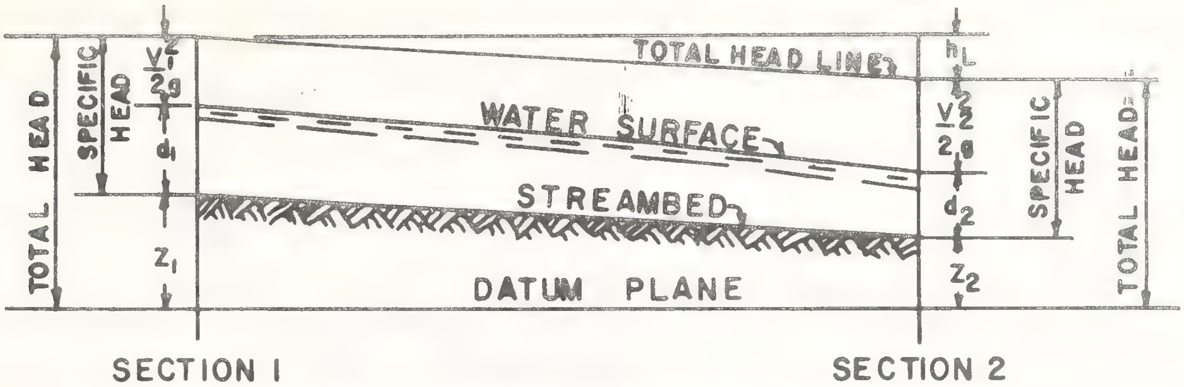


FIGURE 4.43 – Water-surface profile of channel with nonuniform flow.

Between sections 1 and 2, the velocity is increasing and the rate of losing energy is, therefore, not constant. This condition could be caused by a channel slope steeper than that needed to overcome frictional resistance or by a change in channel cross section. Thus, the total head line (also called the energy line or energy gradient) is not a straight line. The water surface line in an open channel is sometimes called the hydraulic grade line.

Critical Flow - With a constant discharge passing a cross section, changing the depth of flow causes a different specific head for each depth. If specific head is plotted against depth of flow, the result is a specific-head (energy) diagram. (See Fig. 4.44)

The specific-head curve is asymptotic to the line representing the energy due to depth and the vertical line of zero depth. Examination of Figure 4.44 reveals several important facts. Starting at the upper right of the curve with a large depth and small velocity, the specific head decreases with decrease in depth, reaching a minimum value at depth d_c, known as critical depth, sometimes called the depth of the minimum energy content. Further decrease in depth results in rapid increase in specific head. For any value of specific head except that corresponding to critical depth, there are alternate depths at which the flow

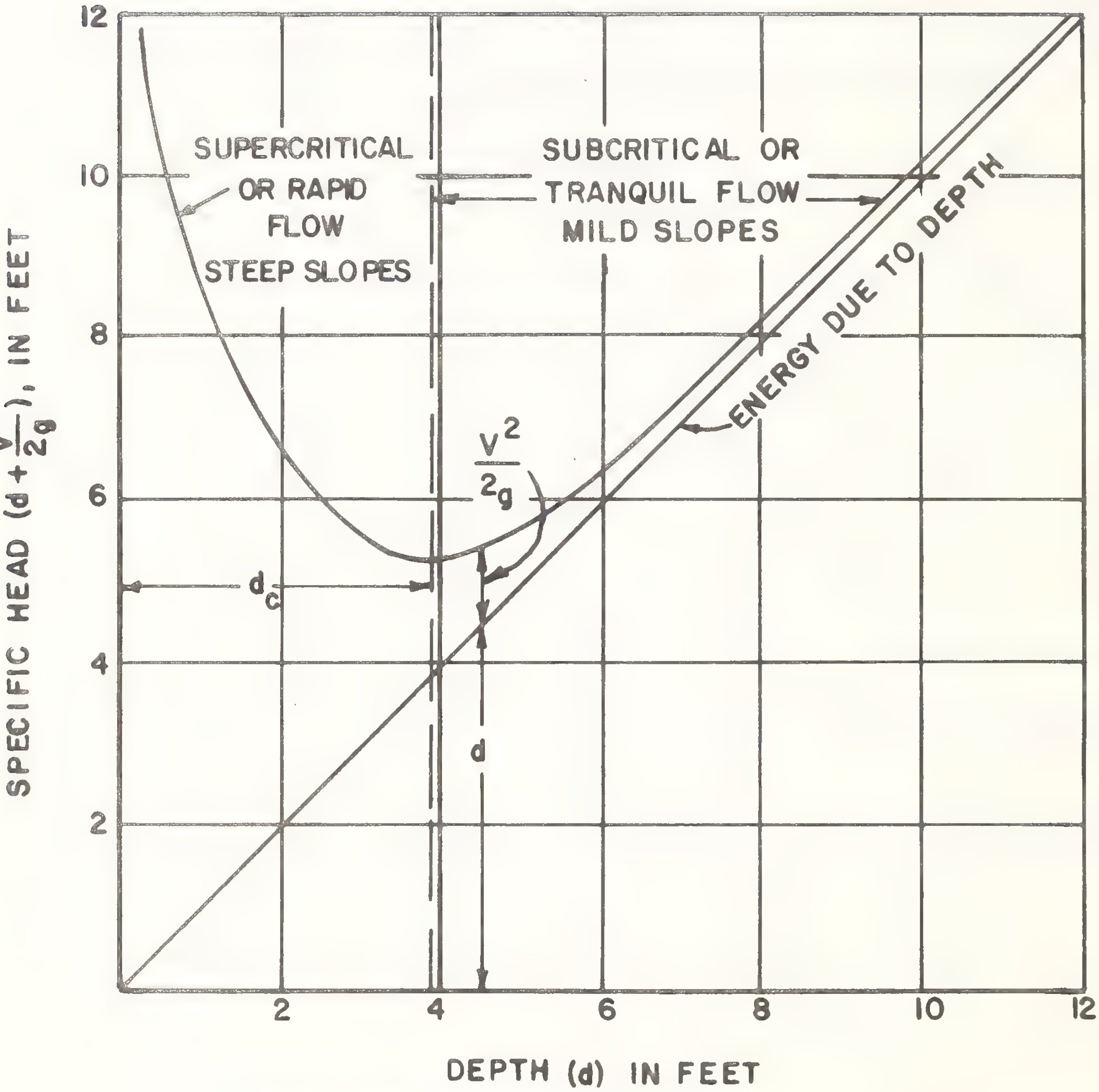


FIGURE 4.44 – Specific head diagram for constant Q.

could occur. These alternate depths are sometimes referred to as equal energy depths.

When the flow occurs at depths greater than critical depth (velocity less than critical), the flow is called subcritical or tranquil. When the flow occurs at depths less than critical depth (velocities greater than critical), the flow is called supercritical, rapid, or shooting. The change from supercritical to subcritical flow is often very abrupt, resulting in the phenomenon known as hydrualic jump. Flow at the critical depth is called critical flow and the velocity at critical depth is the critical velocity. The channel slope which produces critical velocity for given discharge is the critical slope.

Critical depth for a particular discharge is dependent on channel size and shape only and is independent of channel slope and roughness. Critical slope depends upon the channel roughness, the channel geometry and the discharge. For a given critical depth and critical velocity, the critical slope for a particular roughness can be computed by the Manning equation.

Supercritical flow is difficult to control because abrupt changes in alignment or in cross section produce waves which travel downstream, alternating from side to side, and sometimes cause the water to overtop the channel sides. Changes in channel shape, slope, or roughness cannot be reflected upstream except for very short distances (upstream control). Supercritical flow is common in steep flumes, and in mountain streams.

Subcritical flow is relatively easy to control. Changes in channel shape, slope, and roughness affect the flow for some distances upstream (downstream control). Subcritical flow is characteristic of the streams located in the plains and valleys regions where stream slopes are relatively flat.

Critical depth is important in hydraulic analysis because it is always a hydraulic control. The flow must pass through critical depth in going from one type of flow to the other. Typical locations of critical depth are:

(1) At abrupt changes in slope when a flat (subcritical) slope is sharply increased to a steep (supercritical) slope.

(2) At a channel constriction such as a culvert entrance under some conditions.

(3) At the unsubmerged outlet of a culvert or flume on a subcritical slope, discharging into a wide channel or with a free fall at the outlet.

(4) At the crest of an overflow dam or weir.

Distinguishing between the types is important in channel design, thus the location of critical depth and the determination of critical slope for a cross

section of given shape, size, and roughness becomes necessary. When flow occurs at critical depth

$$\frac{A^3}{T} = \frac{Q^2}{g} \tag{12}$$

Critical depth (d_c) can be found from the design charts or computed for various channel cross sections by the following equations:

Rectangular sections

$$d_c = 0.315 \quad \left(\frac{Q}{B}\right)^{2/3} \tag{13}$$

Trapezoidal sections

$$d_c = \frac{4z\ H_o - 3b + \sqrt{16z^2 H_o^2 + 16zH_o b + 9b^2}}{10z} \tag{14}$$

The tables in the "Handbook of Hydraulics" by King provide a much easier solution for cirtical depth than equation (14).

Triangular sections

$$d_c = 0.574\left(\frac{Q}{Z}\right)^{2/5} \tag{15}$$

Circular sections, approximate solution

$$d_c = 0.325\ \left(\frac{Q}{D}\right)^{2/3} + 0.083D \tag{16}$$

Accurate only when $\frac{d_c}{D}$ lies between 0.3 and 0.9.

Where

A = area of cross section of flow, in square feet.

B = the width of a rectangular channel, in feet.

b = bottom width of a trapezodial channel, in feet.

D = diameter of circular conduit, in feet.

g = acceleration of gravity, 32.2 feet per second.

H_o = specific head in section, in feet (equation 9).

Q = rate of discharge, in cubic feet per second.

T = top width of water surface, in feet.

V = mean velocity of flow, in feet per second.

z = slope of sides of a channel (horizontal to vertical)

For irregular sections, critical depth may be found by a trial-and-error solution using equation (12). An expression for the critical velocity (V_c) in channels of any cross section is:

$$V_c = \sqrt{g d_m}$$

Where

$$d_m = \text{the mean depth of flow } (\frac{A}{T}) \qquad (17)$$

In a given channel when the velocity head ($\frac{V^2}{2g}$) is less than one-half the mean depth, the flow is subcritical; if the velocity head is equal to one-half (1/2) the mean depth, the flow is critical; and if the velocity head is greater than one-half (1/2) the mean depth, the flow is supercritical.

Uniform flow within about 10 percent of critical depth is unstable and should be avoided in design. The reason for the unstable flow can be seen by referring to Figure 4.43. As the flow approaches the critical depth from either limb of the curve, a very small change in energy is required for the depth to abruptly change to the alternate depth on the opposite limb of the specific-head curve. If the unstable flow region cannot be avoided in design, the least favorable type of flow should be assumed for the design.

Nonuniform Flow - Truly uniform flow rarely exists in either natural or man-made channels, because changes in channel section, slope, or roughness cause the depths and average velocities of flow to vary from point to point along the channel, and the water surface will not be parallel to the streambed. Flow that varies in depth and velocity along the channel is called nonuniform. The relationship between velocity and cross sectional area is given by:

$$Q = A_1 V_1 = A_2 V_2 \; . \; . \; . \; . \; . \; . \; A_n V_n \qquad (18)$$

This equation is called the continuity equation. Although moderate nonuniform flow actually exists in a generally uniform channel, it is usually treated as uniform flow in such cases. Uniform flow characteristics can readily be computed and the computed values are usually close enough to the actual for all practical

purposes. The types of nonuniform flow are innumerable, but certain characteristic types are described in the following paragraphs. Briefly discussed are the characteristics of nonuniform flow, both subcritical and supercritical, together with common types of nonuniform flow encountered in highway drainage design.

With subcritical flow, a change in channel shape, slope, or roughness affects the flow for a considerable distance upstream, and thus the flow is said to be under downstream control. If an obstruction, such as a culvert, causes ponding, the water surface above the obstruction will be a smooth curve asymptotic to the normal water surface upstream and to the pool level downstream (see Fig. 4.45).

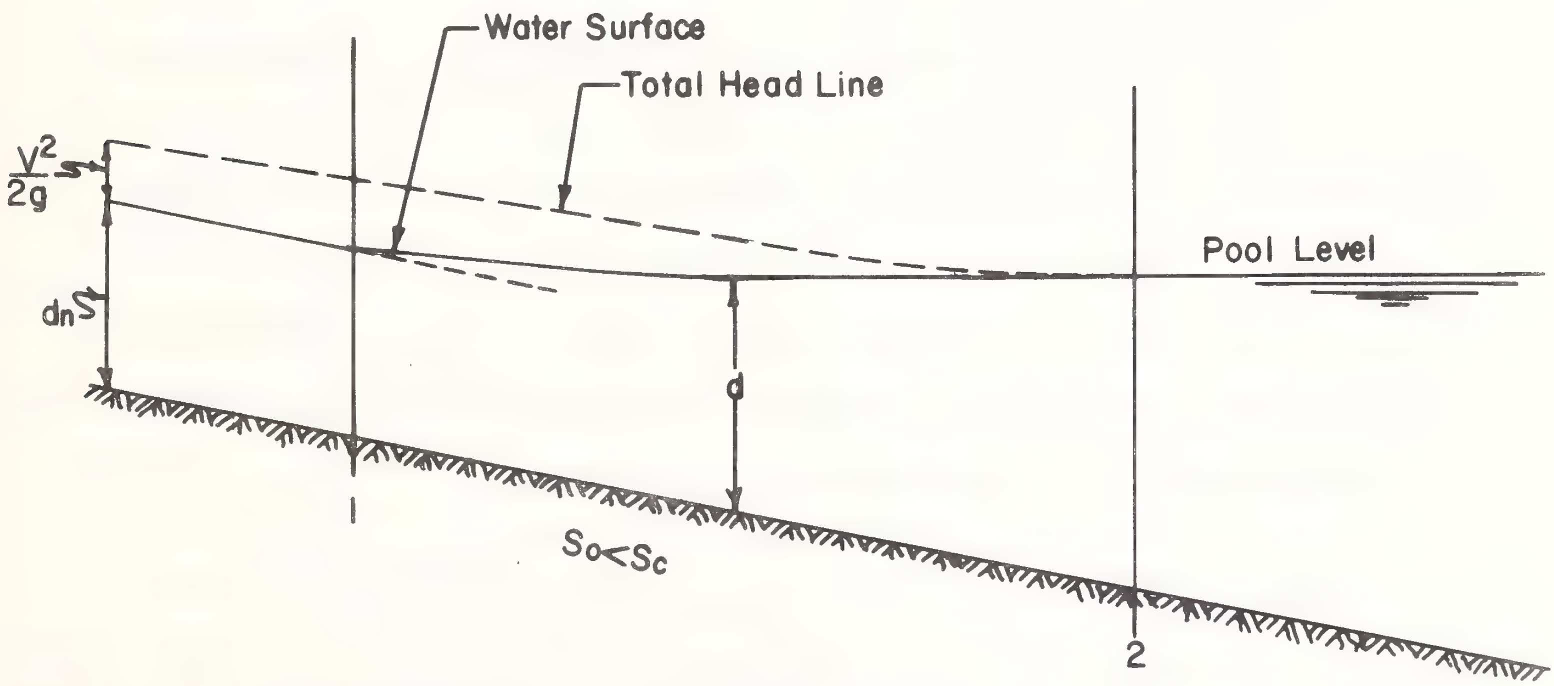


Figure 4.45—Water-surface profile in flow from a channel to a pool.

Another example of downstream control occurs where an abrupt channel enlargement, as at the end of a culvert not flowing full, or a break in grade from a mild to a steep slope, causes a drawdown in the flow profile to critical depth. The water surface profile upstream from a change in section or a break in channel slope will be asymptotic to the normal water surface upstream, but will drop away from the normal water surface on approaching the channel change or break in slope. In these two examples, the flow is nonuniform because of the changing

water depth caused by changes in the channel slope or channel section. Direct solution of open-channel flow by the Manning equation or by the charts in this section is not possible in the vicinity of the changes in the channel section or channel slope.

With supercritical flow, a change in channel shape, slope, or roughness cannot be reflected upstream except for very short distances. However, the change may affect the depth of flow at downstream points; thus, the flow is said to be under upstream control. An example is the flow in a steep chute where the water surface profile draws down from critical depth at the chute entrance and approaches the lesser normal depth in the chute (see Fig. 4.46)

Most problems in highway drainage do not require the accurate computation of water surface profiles; however, the designer should know that the depth in a given channel may be influenced by conditions either upstream or downstream, depending on whether the slope is steep (supercirtical) or mild (subcritical). Three typical examples of nonuniform flow are shown in Figures 4.44 - 4.46 and are discussed in the following paragraphs. The discussion also explains the use of the total head line in analyzing nonuniform flow.

Figure 4.45 shows a channel on a mild slope, discharging into a pool. The vertical scale is exaggerated to illustrate the case more clearly. Cross section 1 is located at the end of uniform channel flow in the channel and cross section 2 is located at the beginning of the pool. The depth of flow d between sections 1 and 2 is changing and the flow is nonuniform. The water surface profile between the sections is known as a backwater curve and is characteristically very long. The computation of backwater curves is explained in textbooks and handbooks on hydraulics.

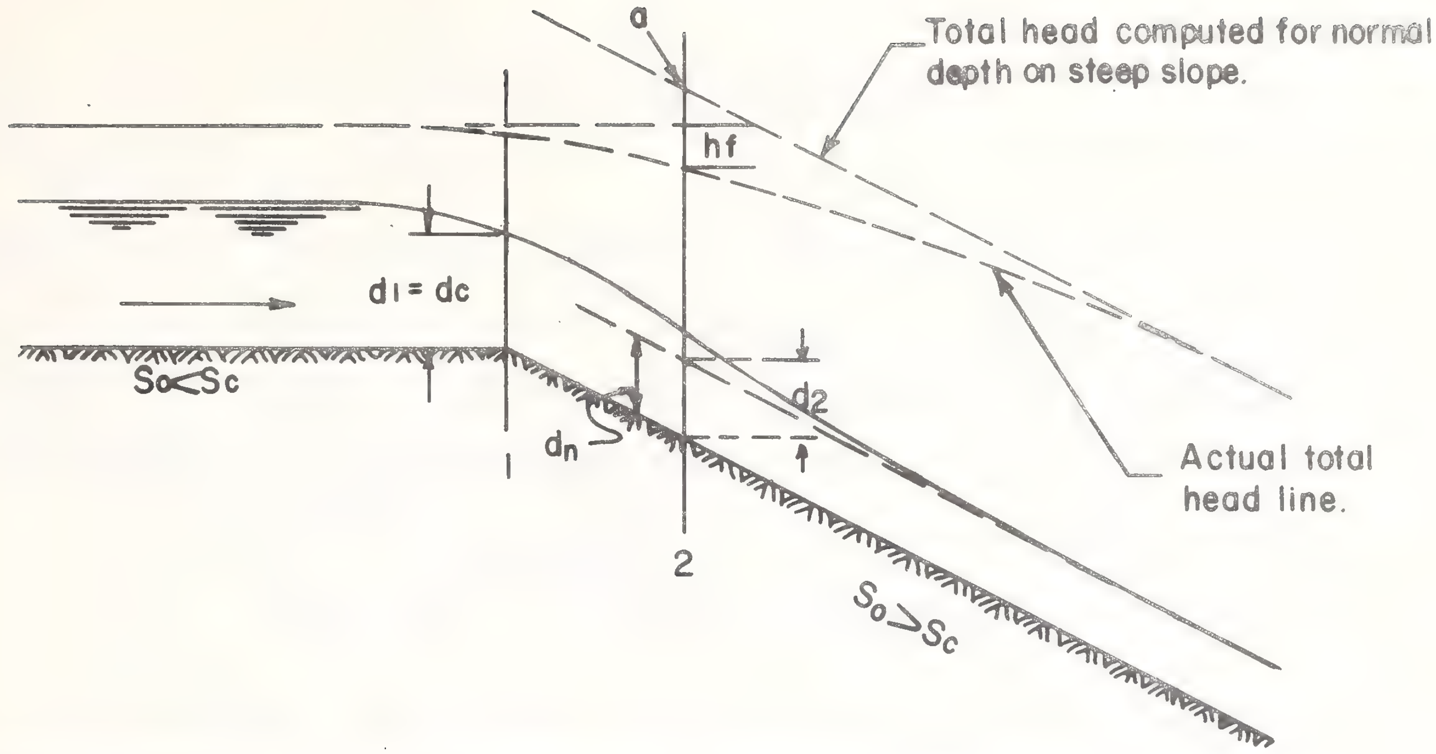


Figure 4.46 - Water-surface profile in changing from subcritical to supercritical channel slope.

Figure 4.46 shows a channel in which the slope changes from subcritical to supercritical. The flow profile passes through critical depth near the break in slope (section 1). This is true whether the upstream slope is mild, as in the sketch, or whether the water above section 1 is ponded, as would be the case if section 1 were the crest of a spillway of a dam. If, at section 2, the total head were computed, assuming normal depth on the steep slope, it would plot (point a on sketch) above the elevation of total head at section 1. This is physically impossible, because the total head line must slope downward in the direction of flow. The actual total head line will take the position shown, and have a slope approximately equal to S_c at section 1 and approaching slope S_o farther downstream. The drop in the total head line h_f between section 1 and 2 represents the loss in energy due to friction. At section 2 the actual depth d_2 is greater than d_n because sufficient acceleration has not occurred and the assumption of normal depth at this point would clearly be in error. As section 2 is downstream, so that the total head for normal depth drops below the pool elevation above section 1, the actual depth quickly approaches the normal depth for the steep channel. This type of water surface curve (section 1 to section 2)

is characteristically much shorter than the backwater curve discussed in the previous paragraph.

Another common type of nonuniform flow is the drawdown curve to critical depth which occurs upstream from section 1 (Fig. 4.46) where the water surface passes through critical depth. The depth gradually increases upstream from critical depth to normal depth provided the channel remains uniform through a sufficient length. The length of the drawdown curve is much longer than the curve from critical depth to normal depth in the steep channel.

Figure 4.46 shows a special case for a steep channel discharging into a pool. A hydraulic jump makes a dynamic transition from the supercritical flow in the steep channel to the subcritical flow in the pool. This situation differs from that shown in Figure 4.44 because the flow approaching the pool in Figure 4.46 is supercritical and the total head in the approach channel is large relative to the pool depth. In general, supercritical flow can be changed to subcritical flow by passing through a hydraulic jump. The violent turbulence in the jump dissipates energy rapidly, causing a sharp drop in the total head line between the supercritical and subcritical states of flow. A jump will occur whenever the ratio of the depth d_1 in the approach channel to the depth d_2 in the downstream channel reaches a specific value. Note in Figure 4.46 that normal depth in the approach channel persists well beyond the point where the projected pool level would intersect the water surface of the channel at normal depth. Normal depth can be assumed to exist on the steep slope upstream from section 1, which is located about at the toe of the jump. More detailed information on the hydraulic jump may be found in textbooks on hydraulics.

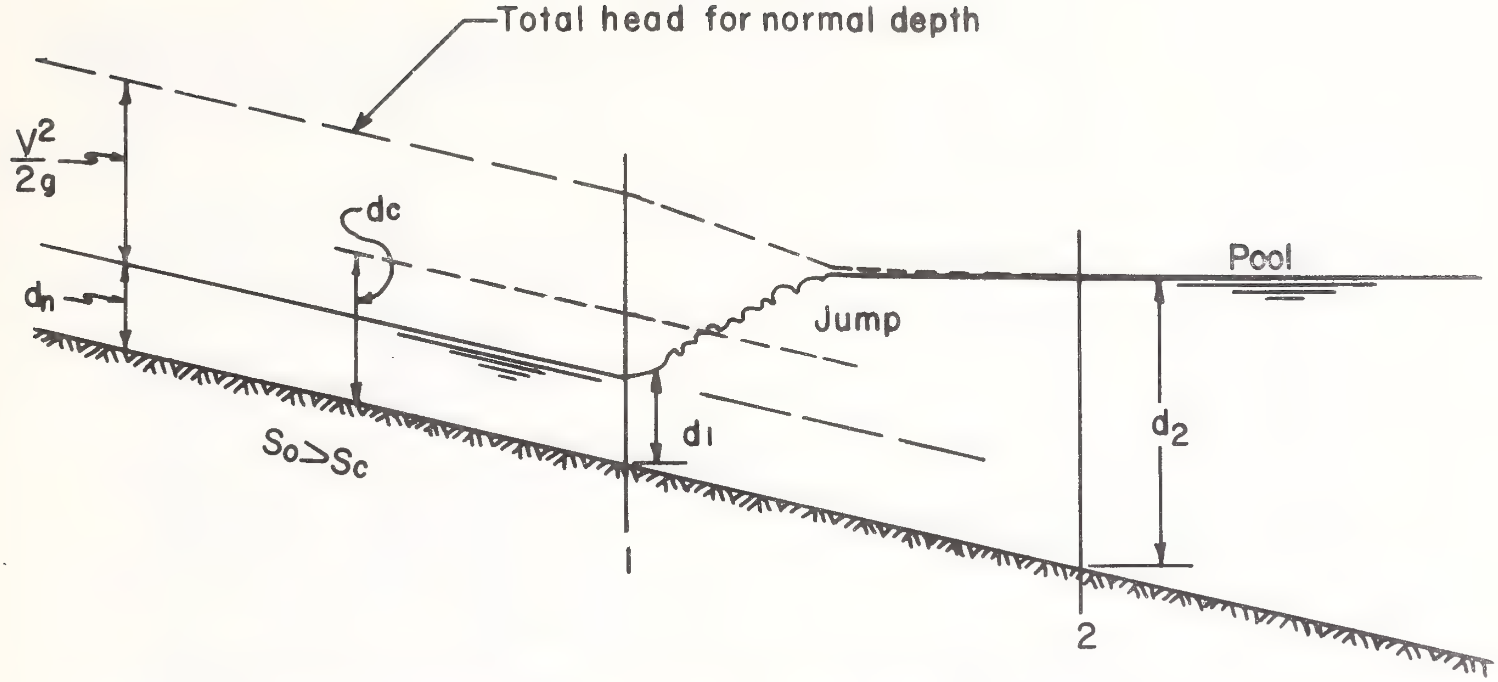


Figure 4.47—Water-surface profile illustrating hydraulic jump.

Table 4.32

Mannings n For Channels, Natural Streams and Pipes

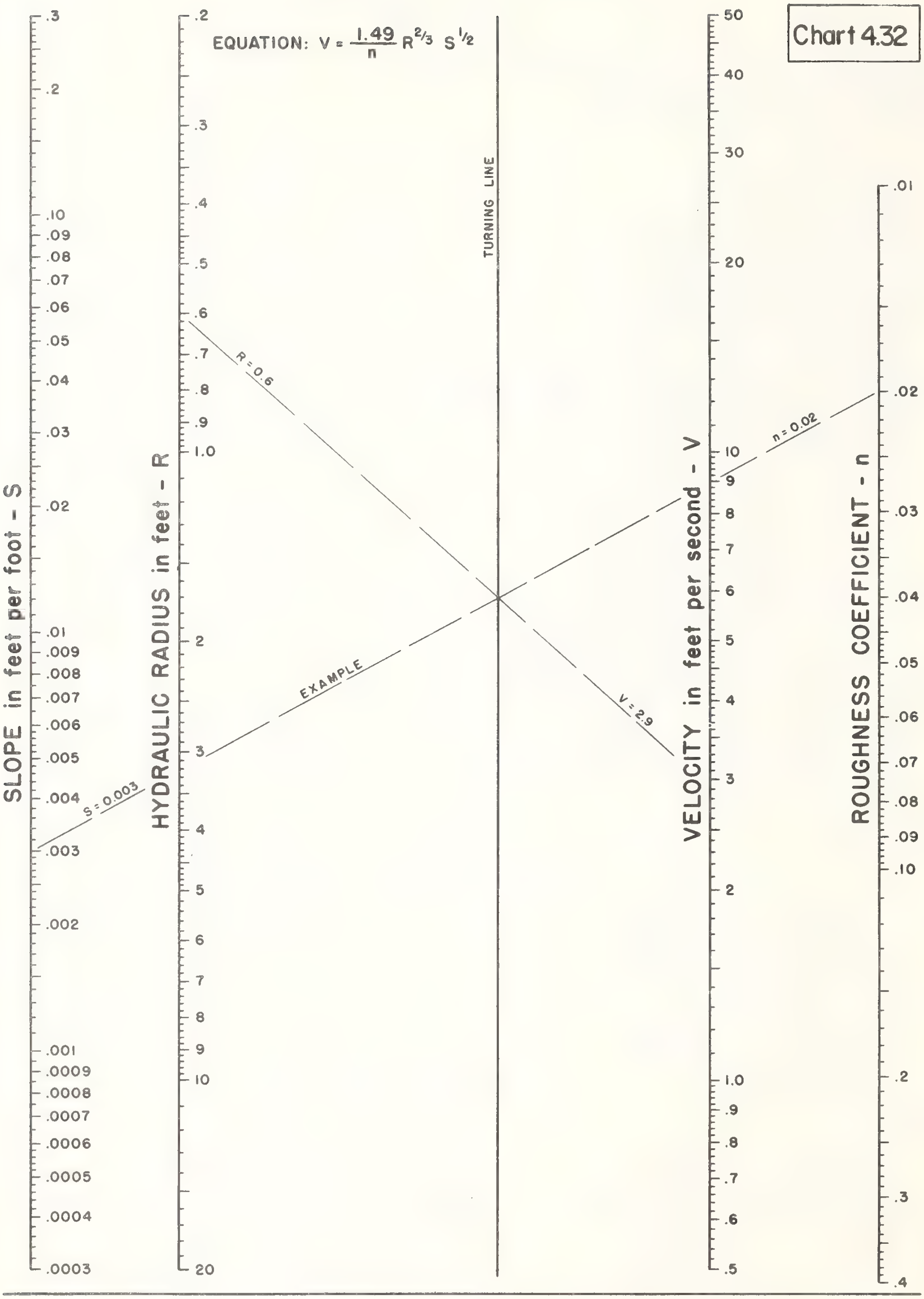
	Range	Ordinary Design Value
Open Channels - lined		
Asphalt		
Smooth		.013
Rough		.016
Concrete		
Smooth forms or troweled	.011-.015	.013
Float finish	.013-.016	.014
Unfinished	.014-.020	.017
Gunite, good		.019
Gunite, wavy		.022
Cemeted rubble masonry	.017-.030	.025
Dressed ashlar masonry	.013-.017	.015
Buck in Mortar	.012-.018	.015
Open Channels		
Excavated		
Earth, uniform and straight		
clean, neat-new to weathered	.016-.022	
short grass, few weeds	.022-.027	
gravelly soil, clean	.022-.030	
Earth, winding		
Clean	.023-.030	
Grass, some weeds	.025-.033	
Dense weeds or aquatic plants	.030-.040	
Stony bottom and weedy banks	.025-.040	
Dragline Excavated		
No vegetation	.025-.030	
Light brush on banks	.035-.060	
Channels not maintained, weeds and brush uncut		
Dense weeds to flow depth	.050-.120	
Clean bottom, brush on banks	.040-.080	
Dense brush, high stage	.080-.140	
Rock cuts		
Fairly smooth and uniform	.025-.040	
Rather irregular	.035-.050	
Natural Streams (top width at flood stage 100 ft)		
Streams on prairie, plain, or valley floor		
Fairly regular section		
Clean, straight, no rifts or deep pools	.025-.033	.030
Clean, winding, some pools and shoals	.033-.045	.040
Clean, winding, some weeds and stones	.035-.050	.045
Clean, winding, more weeds and stones	.045-.060	.050
Sluggish reaches, weedy deep pools	.050-.080	.070
Floodways with heavy stand timber underbrush	.075-.150	.100
Mountain Streams with trees and brush along banks submerged at high stages		
Bottom gravel, cobbles, and few boulders	.030-.050	.040
Bottom with large boulders	.040-.070	.050

Table 4.32 (Cont'd)

	Range	Ordinary Design Values
Flood Plains (adjacent to natural streams)		
Pasture, short to high grass	.030-.050	
Cultivated areas		
no crop	.030-.040	
Mature row crops	.025-.045	
Mature field crops	.030-.050	
Heavy Weeds, scattered brush	.035-.070	
Light brush and trees		
winter to summer	.05 - .08	
Medium brush and trees		
winter to summer	.07 - .16	
Dense willows, summer, not bent over	.15 - 2.0	
Cleared land with tree stumps, 100-150		
no sprouts	.04 - .05	
heavy growth sprouts	.16 - .08	
Heavy stand timber, some sown little brush or undergrowth		
water below branches	.10 - .12	
water reaching branches	.12 - .16	
Major Streams (surface at flood stage 100 ft) n value usually less than for minor streams of like description because banks offer relatively less resistance		
Large stream, regular section, no branch or boulders	.025-.060	
Large stream, irregular with boulders and some brush	.035-.100	
Pipes		
Concrete	.011-.015	.013
CSP		
2 2/3" x 1/2" Corr.		
Unpaved		.024
25 percent paved		.021
Fully paved		.012
3" x 1" Corr.		
Unpaved		.027
25 Percent paved		.023
Fully paved		.012
Helical		
Unpaved	.012-.020	*
25 percent paved	.015-.020	*
Fully paved	.012-.012	.012
Structural Plate Pipe	.030-.033	*

* N. values Varies with size of Pipes, refer to Manufacturers Specifications.

Chart 4.32



NOMOGRAPH FOR SOLUTION OF MANNING EQUATION

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4.62 RECTANGULAR, TRAPEZOIDAL, AND TRIANGULAR CHANNELS

Description of Charts - Charts 4.33 - 4.61 are designed for use in the direct solution of the Manning equation for various sized open channels of rectangular, trapezoidal, and triangular cross section. Each chart (except the triangular cross section channel Chart No. 4.61) is prepared for a channel of given bottom width, and having a particular value of Manning's n, but auxiliary scales make the charts applicable to other values of n.

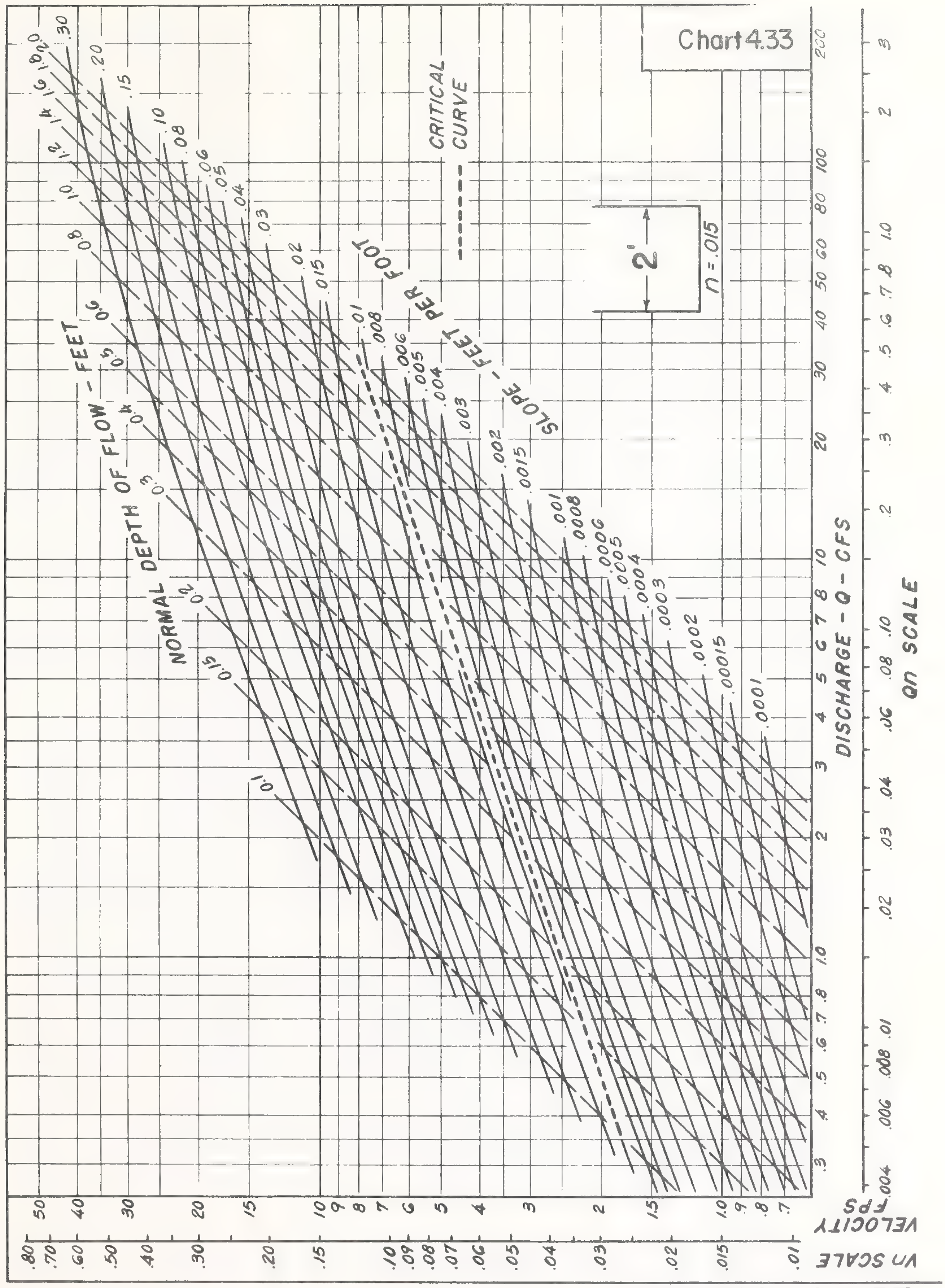
The rectangular cross section channel Charts, Nos. 4.33 - 4.46 are prepared for an n of 0.015 (average value for concrete). A separate chart is provided for each foot of width from 2 feet to 10 feet and for each even foot of width from 10 feet to 20 feet.

The trapezoidal cross section channel Charts, Nos. 4.47 - 4.60, are prepared for an n of 0.03 and side slopes of 2:1 (horizontal to vertical). A separate chart is provided for each foot of bottom width from 2 feet to 10 feet and for each even foot of width from 10 feet to 20 feet.

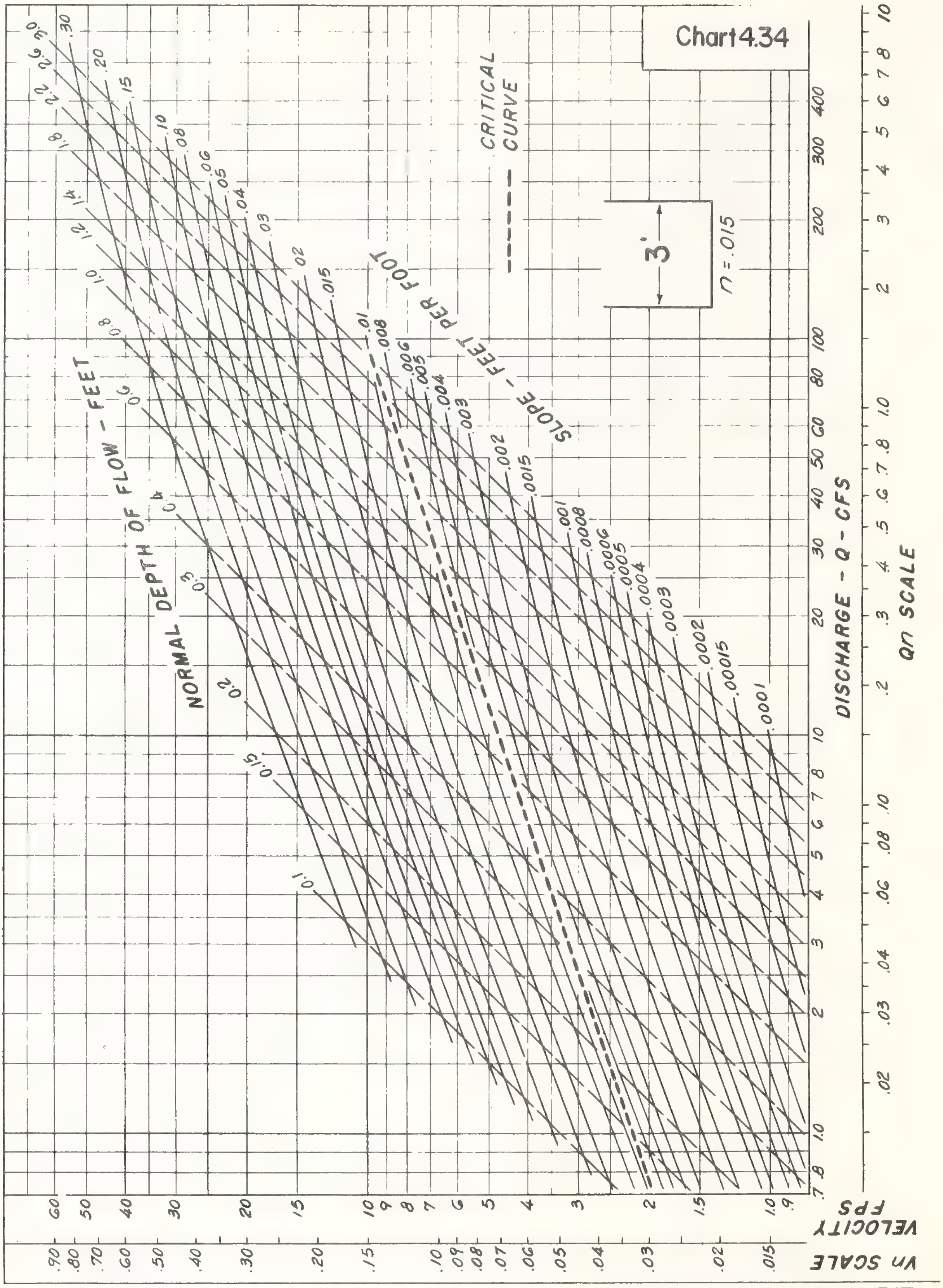
The charts for rectangular and trapezoidal cross section channels are similar in design and method of use. The abscissa scale is discharge, in cubic feet per second (c.f.s.) and the ordinate scale is velocity, in feet per second (f.p.s.). Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (in feet), and slightly inclined lines representing channel slope (in feet per foot). A heavy dashed line on each chart shows the position of critical flow. Auxiliary abscissa and ordinate scale are provided for use with values of n other than those values used in preparing the chart.

In these charts, and subsequent ones similarly designed, interpolations may be made with confidence, not only on the ordinate and abscissa scales, but between the inclined lines representing depth and slope.

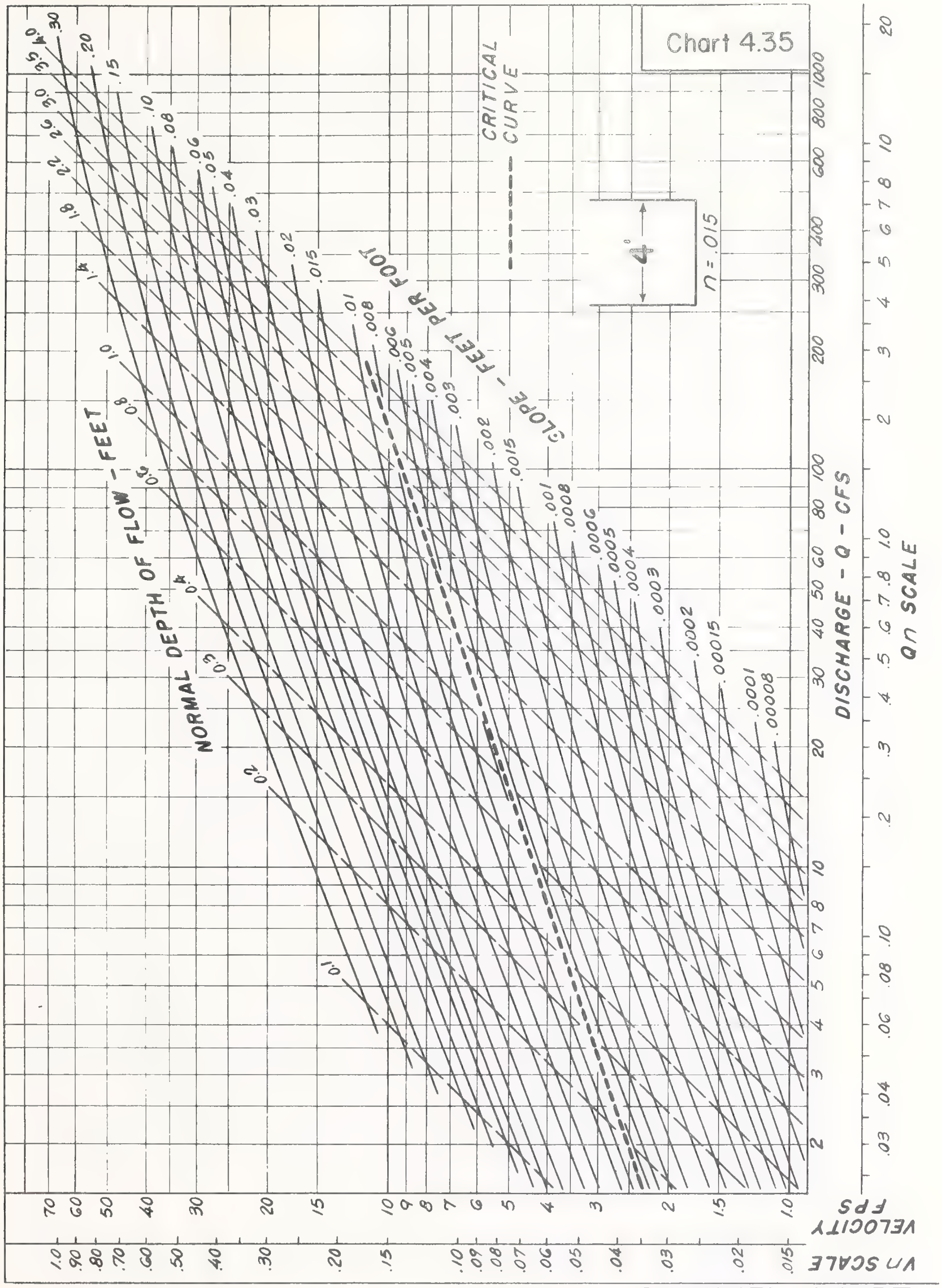
The triangular cross section channel Chart, No. 4.61, is prepared in nomograph form. It may be used for street sections with a vertical (or nearly vertical) curb face. (The curbed street section is a triangular section with one leg vertical). The equation given on the chart ignores the resistance of the curb face, but this resistance is negligible from a practical viewpoint, provided the width of flow is at least 10 times the depth of the curb face; that is, if $Z > 10$. The equation gives a discharge about 19 percent greater than will be obtained by the common procedure of computing discharge from the hydraulic radius of the entire section. The latter procedure is not recommended for shallow flow with continuously varying depth. The nomograph may also be used for shallow V-shaped sections by following the instruction on the chart.



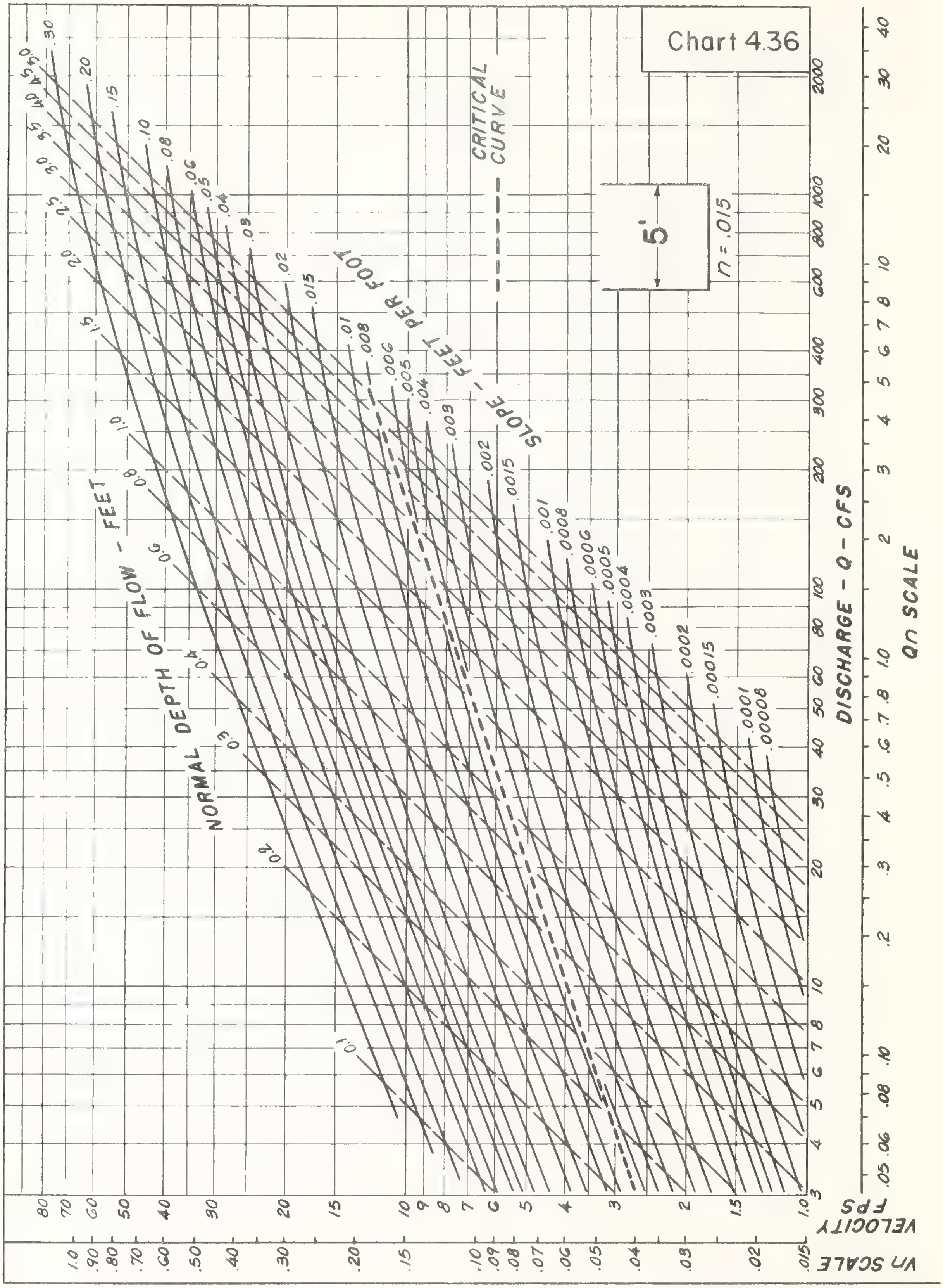
CHANNEL CHART
VERTICAL b = 2 FT.



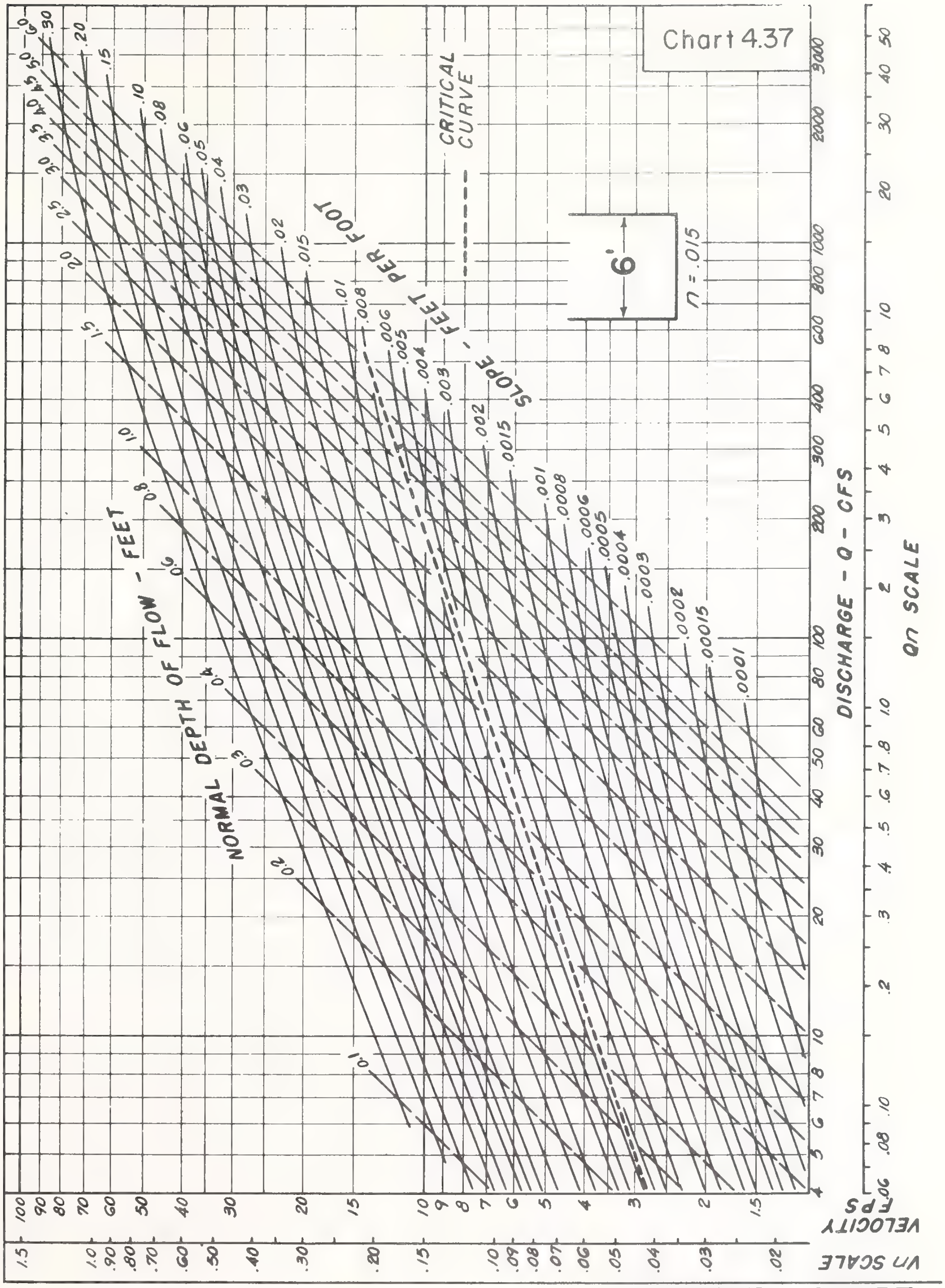
CHANNEL CHART
VERTICAL b = 3 FT.



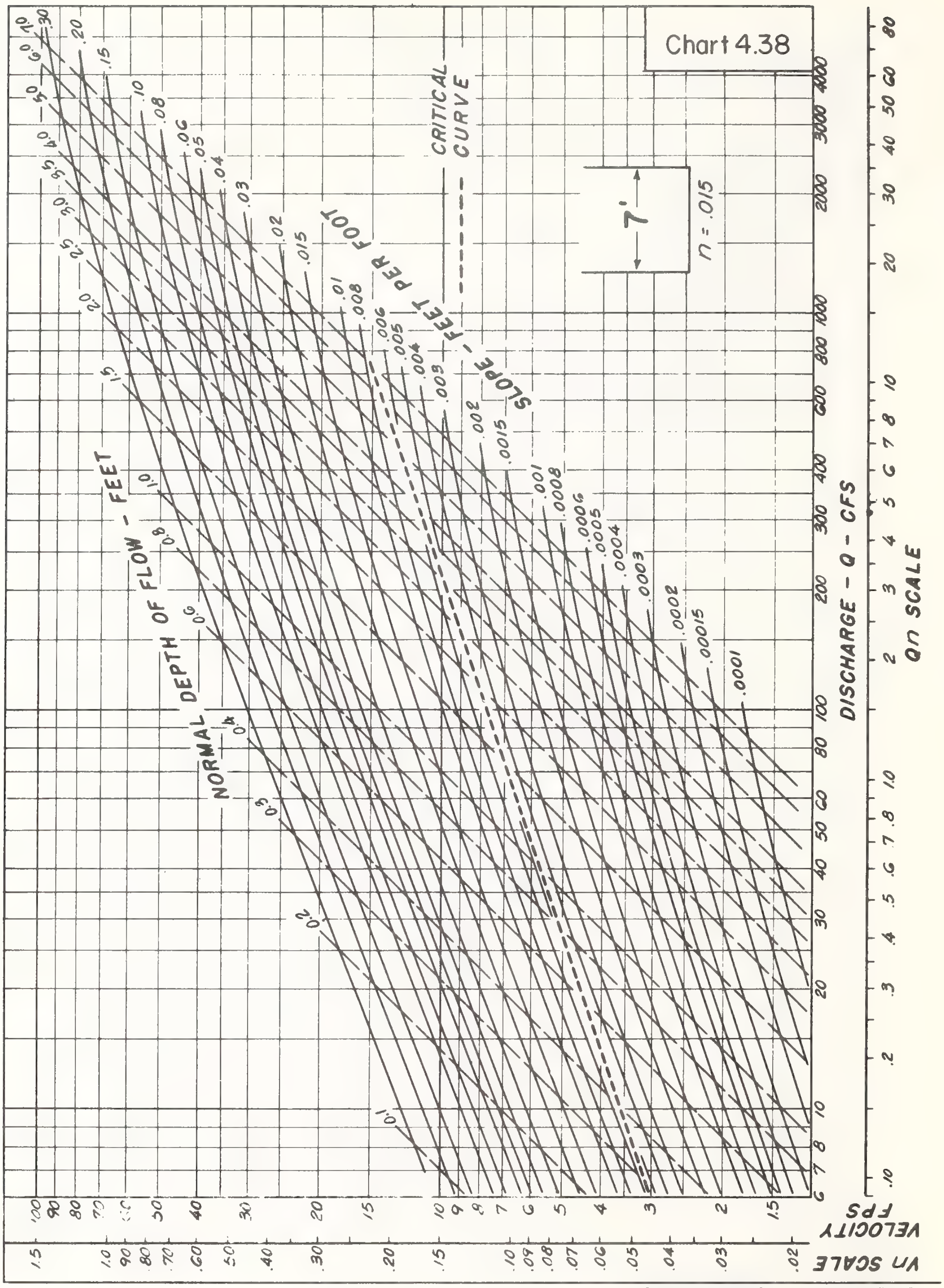
CHANNEL CHART
VERTICAL b = 4 FT.



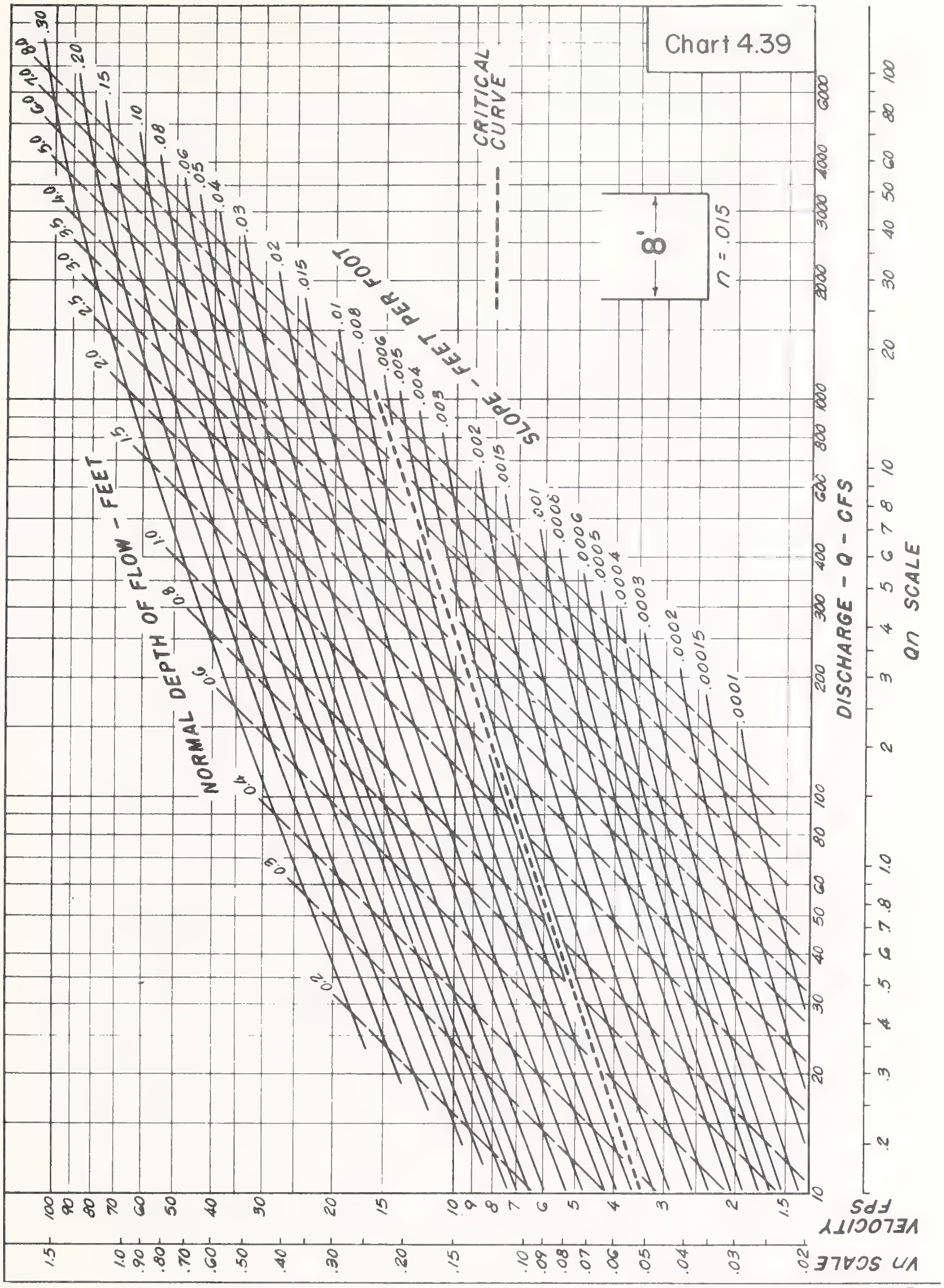
CHANNEL CHART
VERTICAL b = 5 FT.



CHANNEL CHART
VERTICAL b = 6 FT.



CHANNEL CHART
VERTICAL b = 7 FT.



CHANNEL CHART
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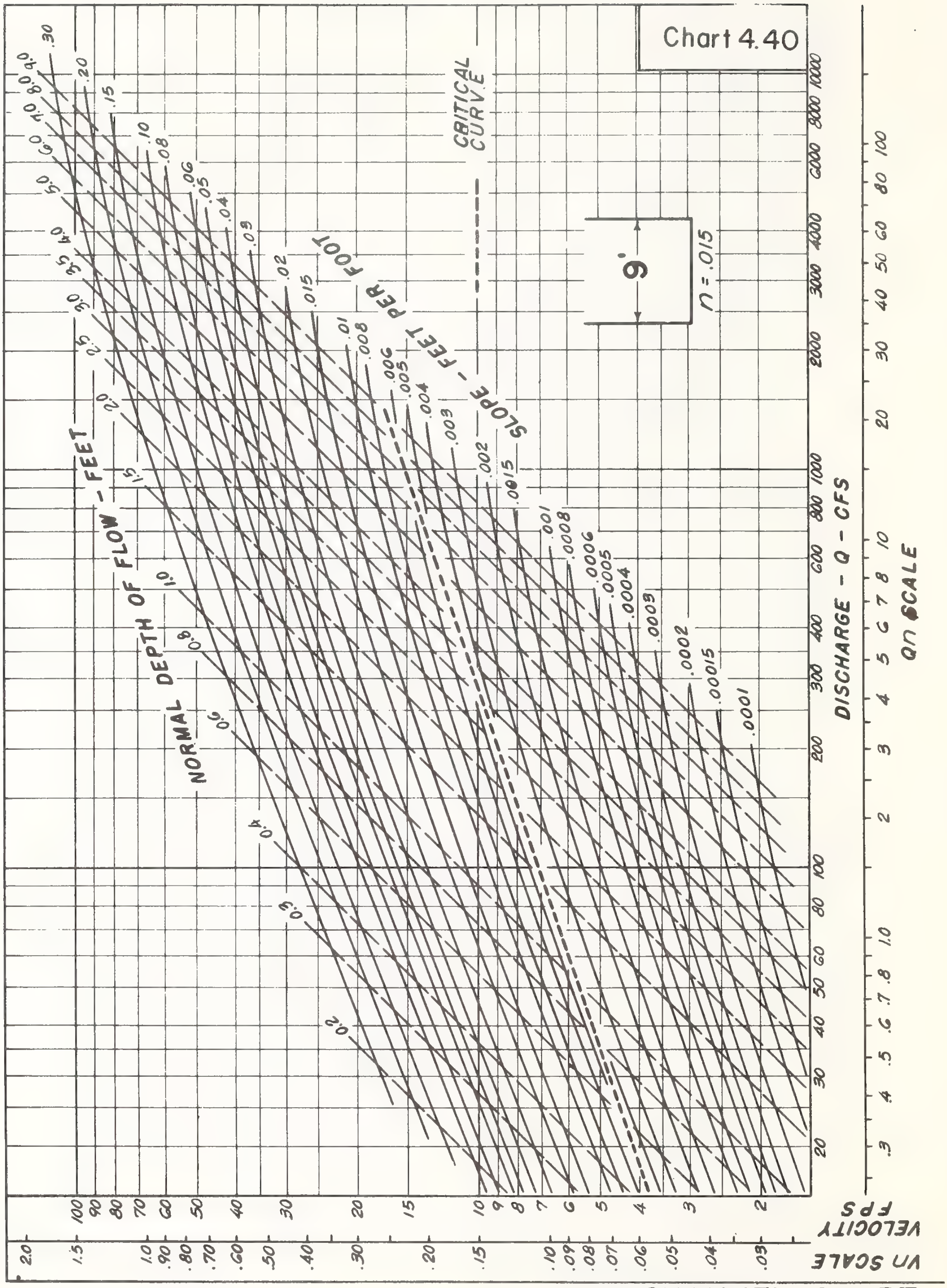

CHANNEL CHART
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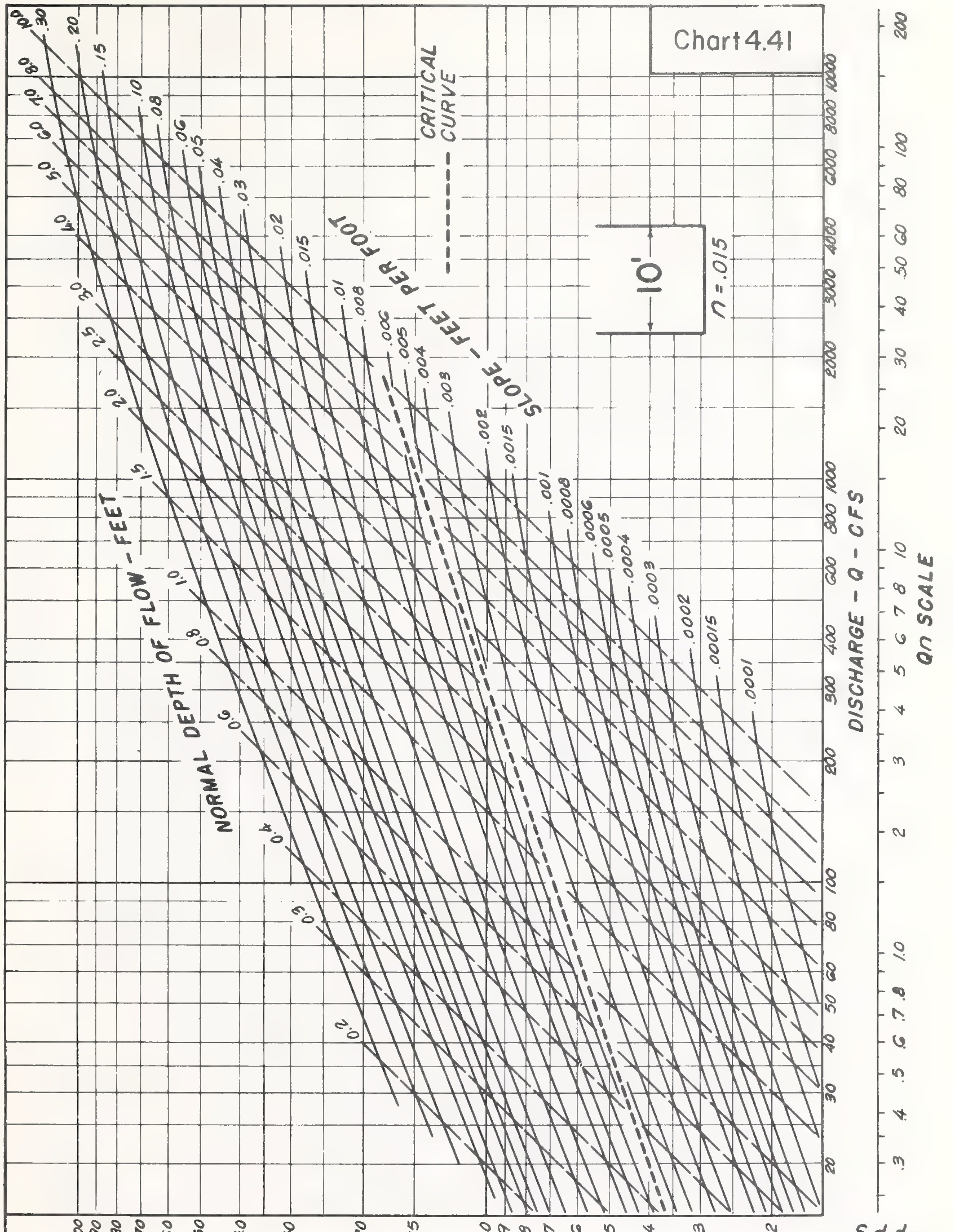

CHANNEL CHART
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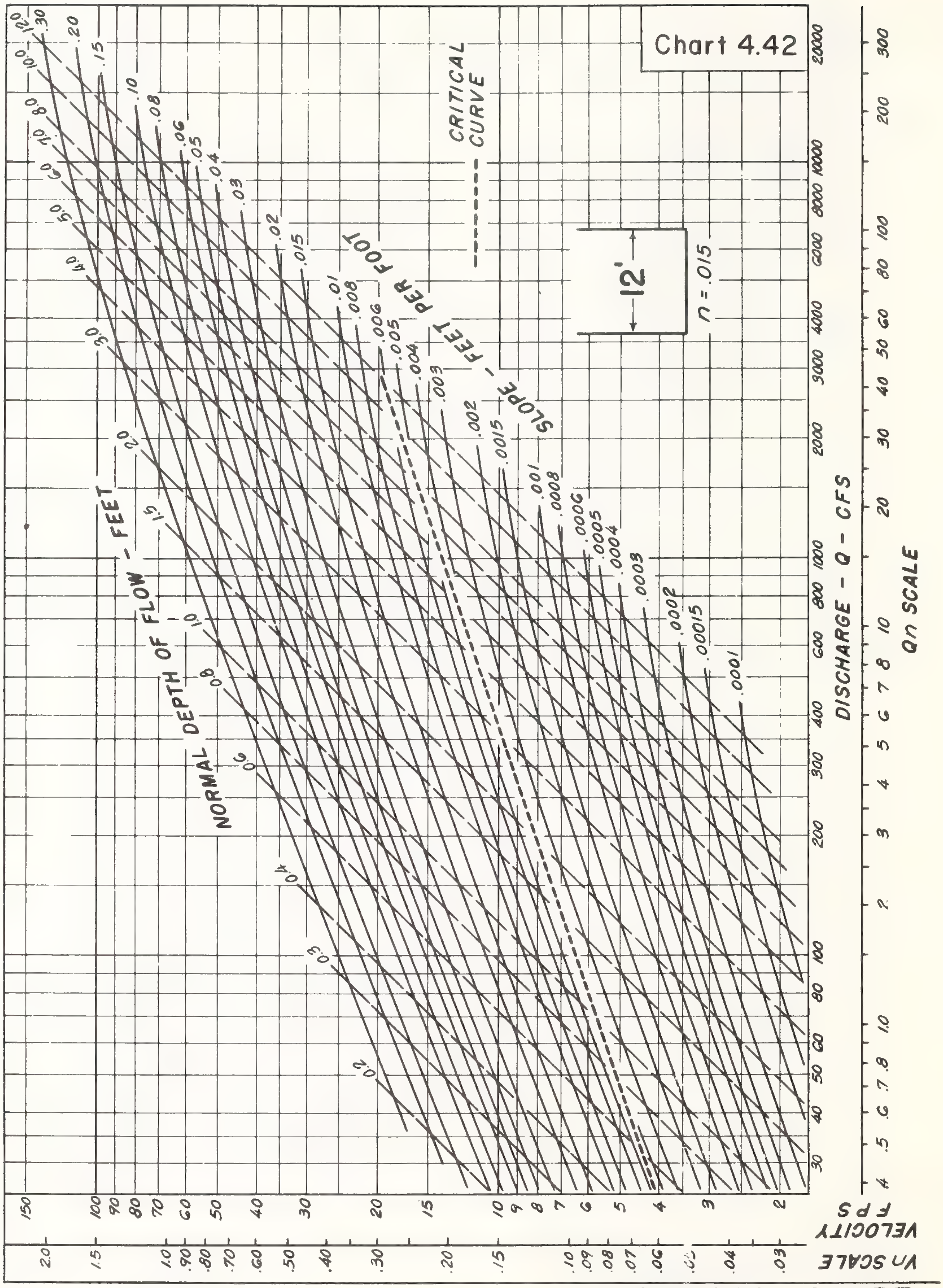
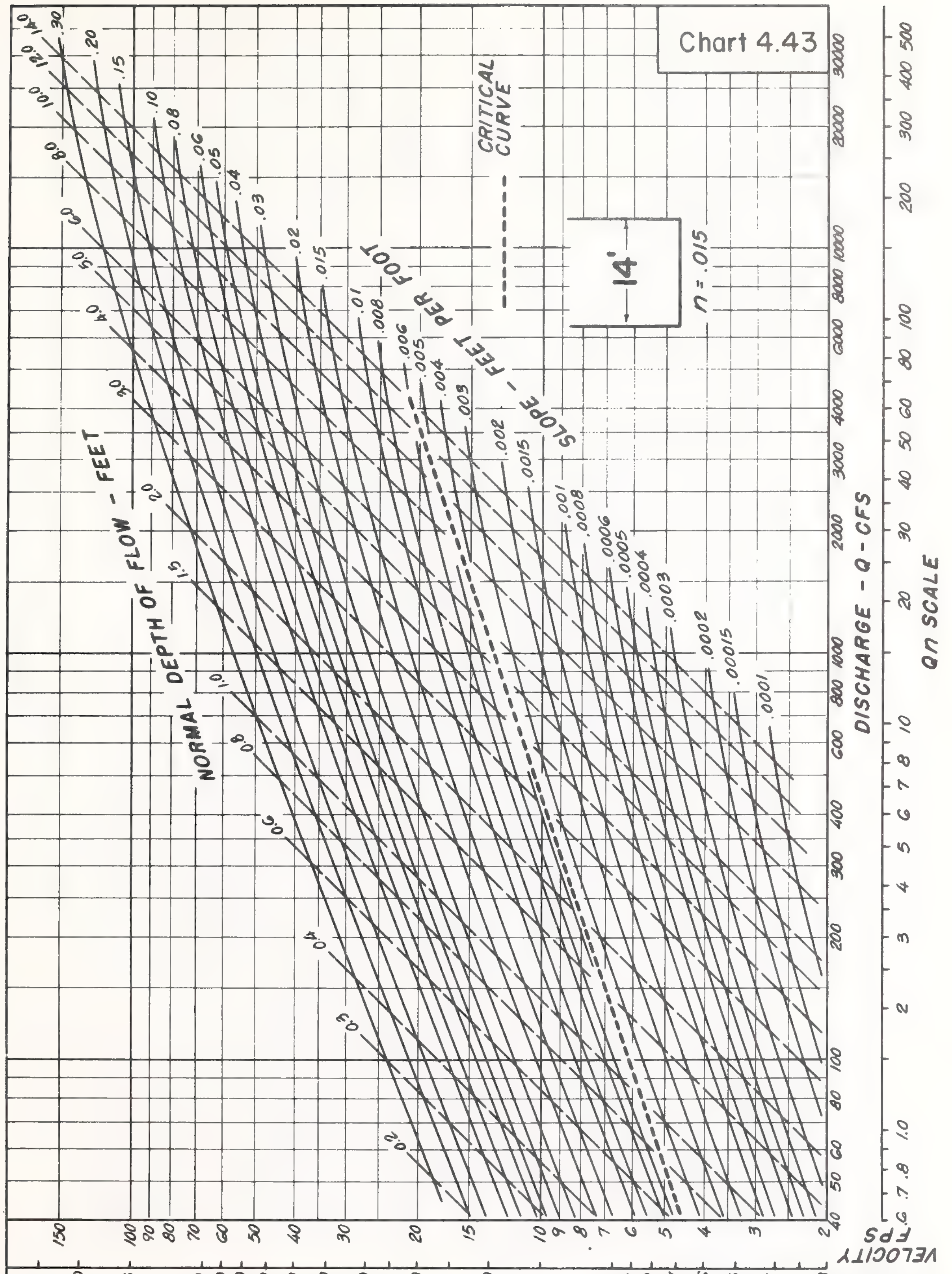
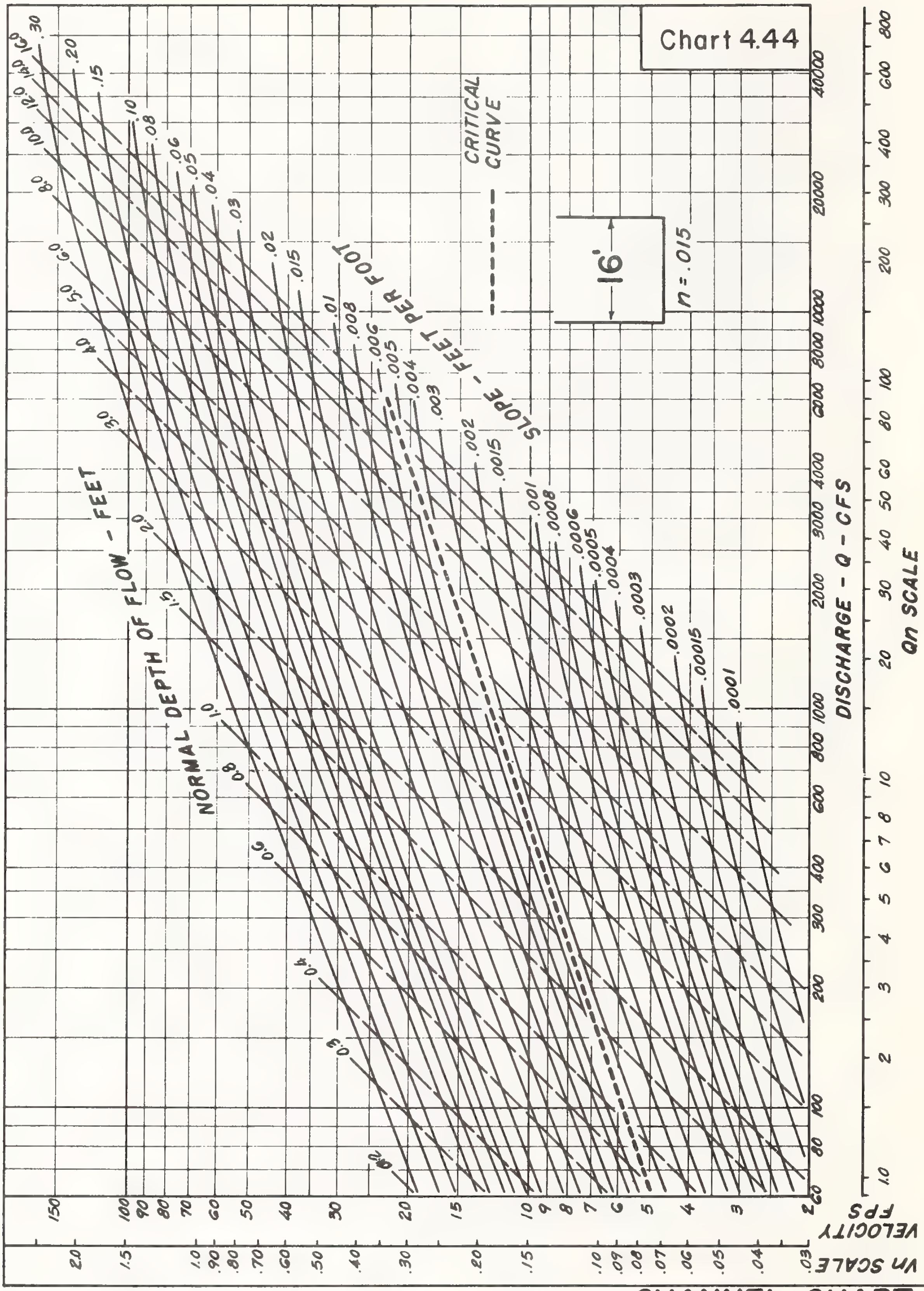


CHANNEL CHART
VERTICAL b = 12 FT.



CHANNEL CHART
VERTICAL b = 14 FT.



CHANNEL CHART
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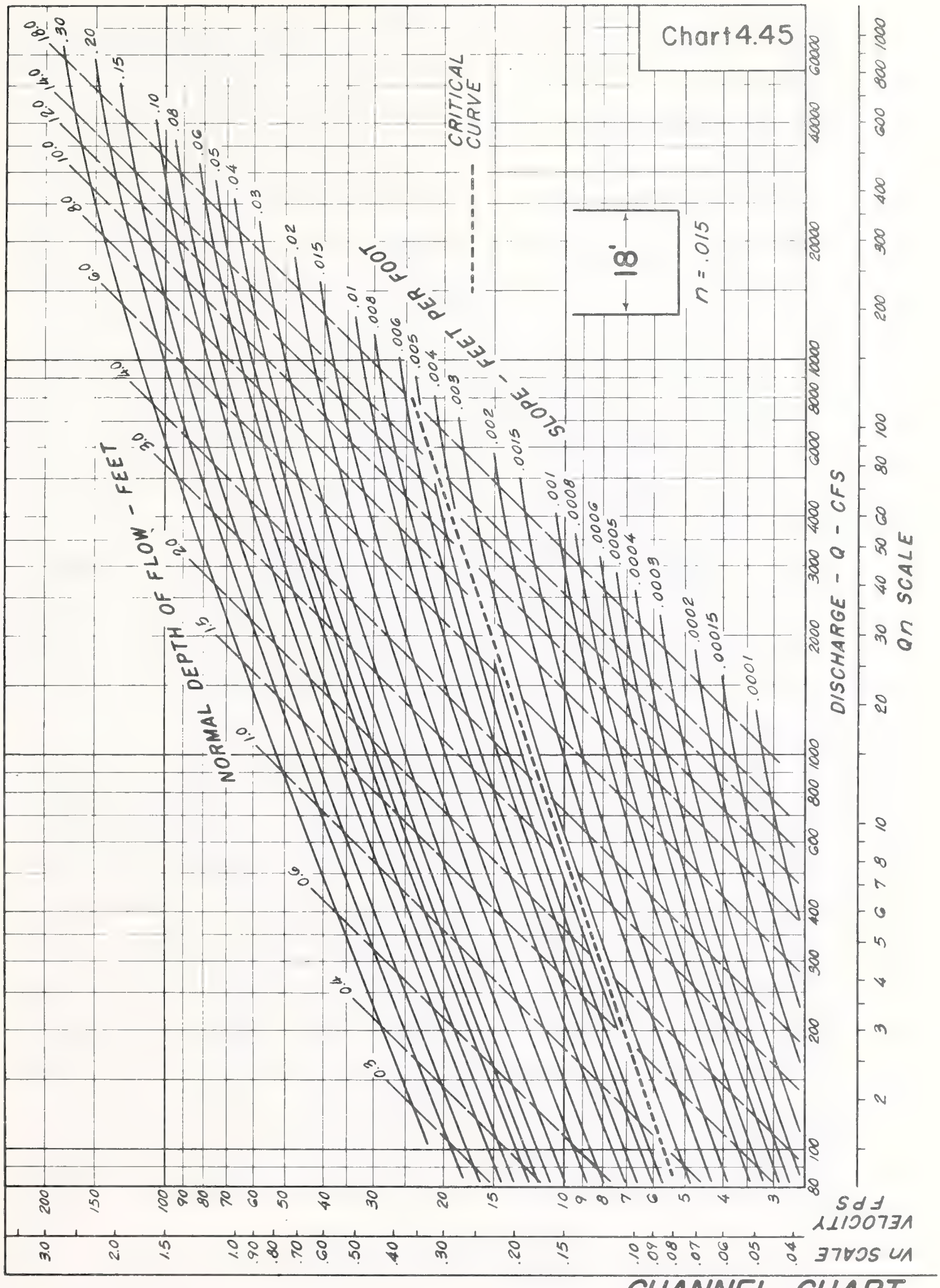

CHANNEL CHART
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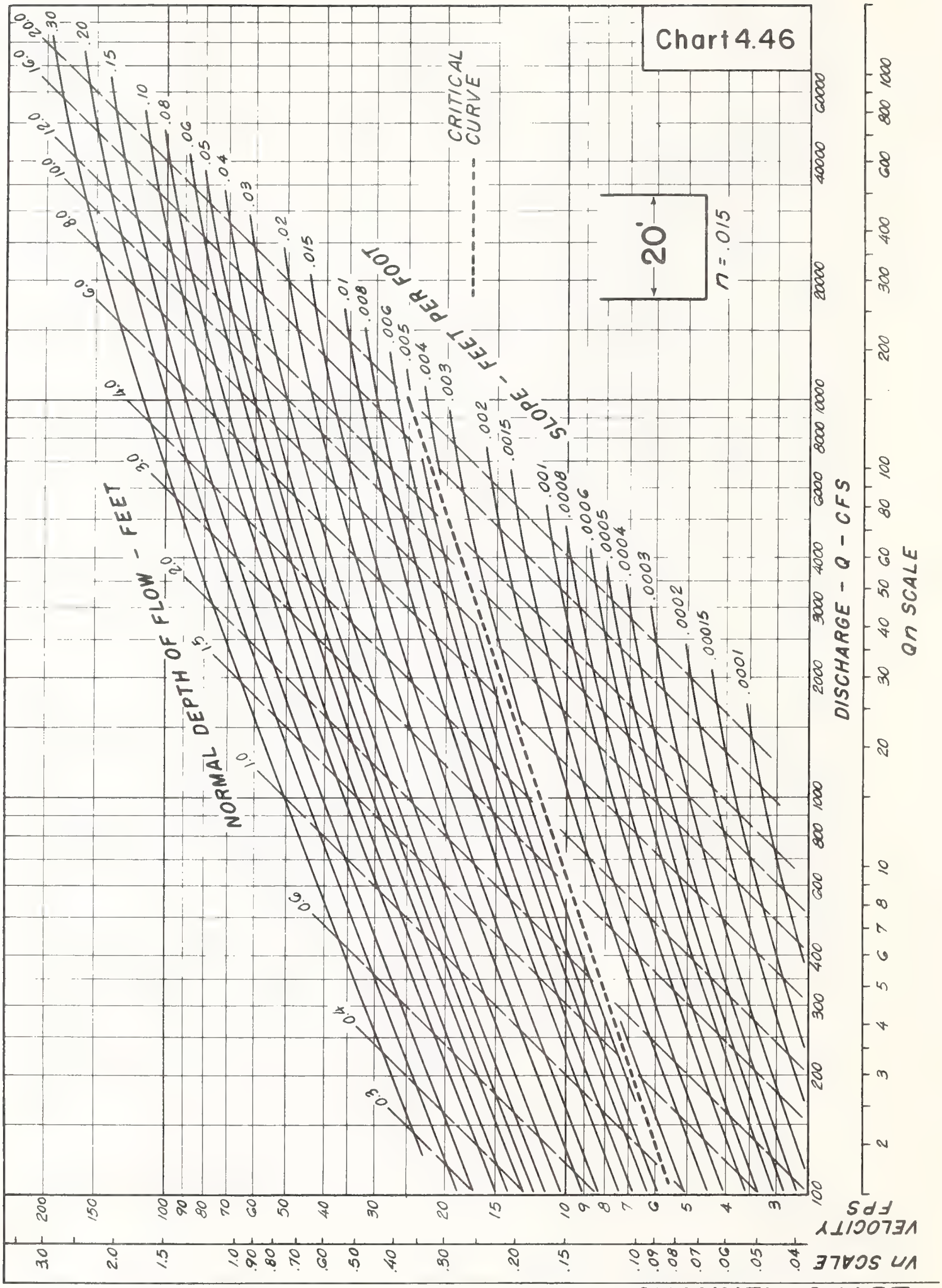
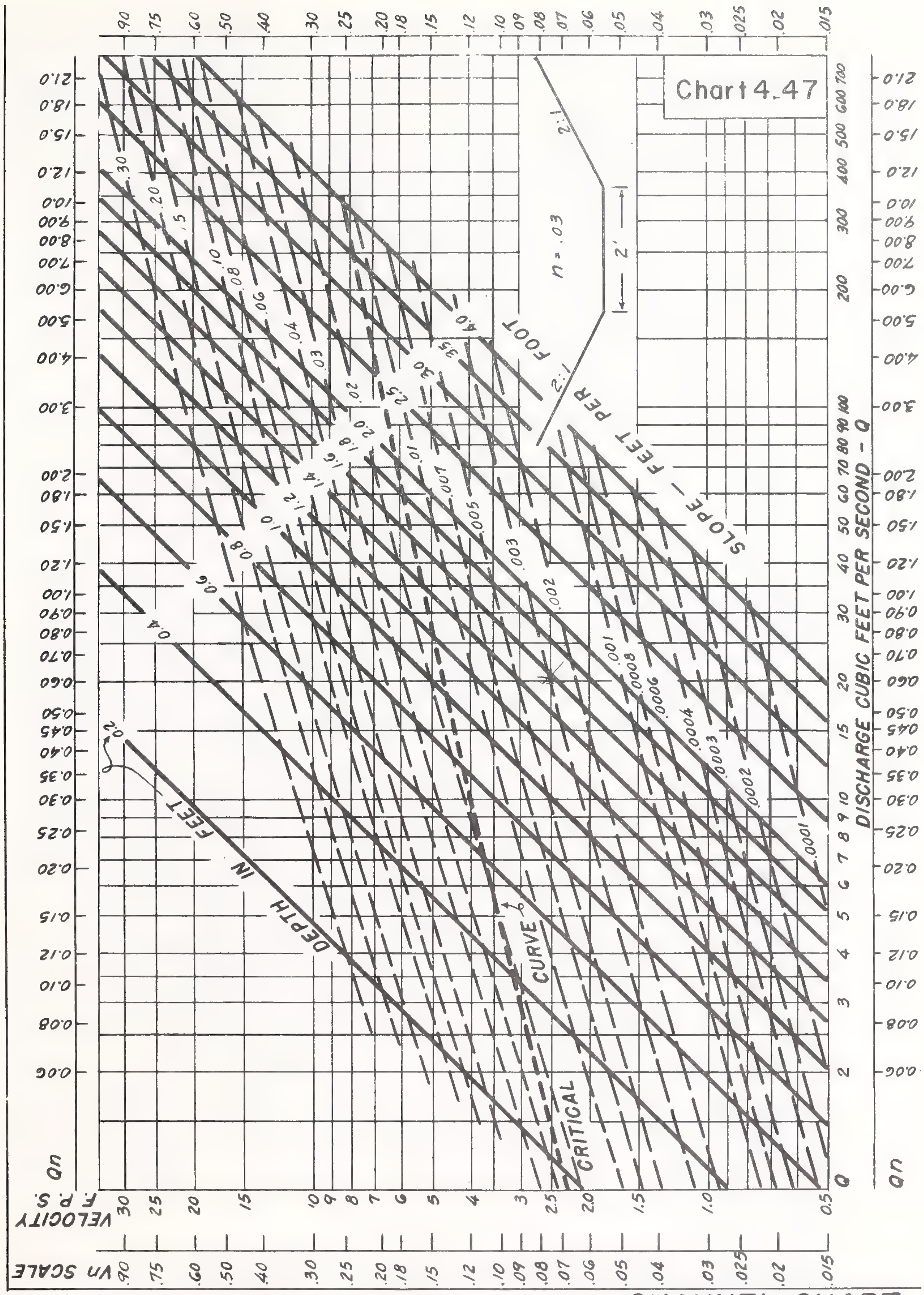


CHANNEL CHART
VERTICAL b = 20 FT.



CHANNEL CHART
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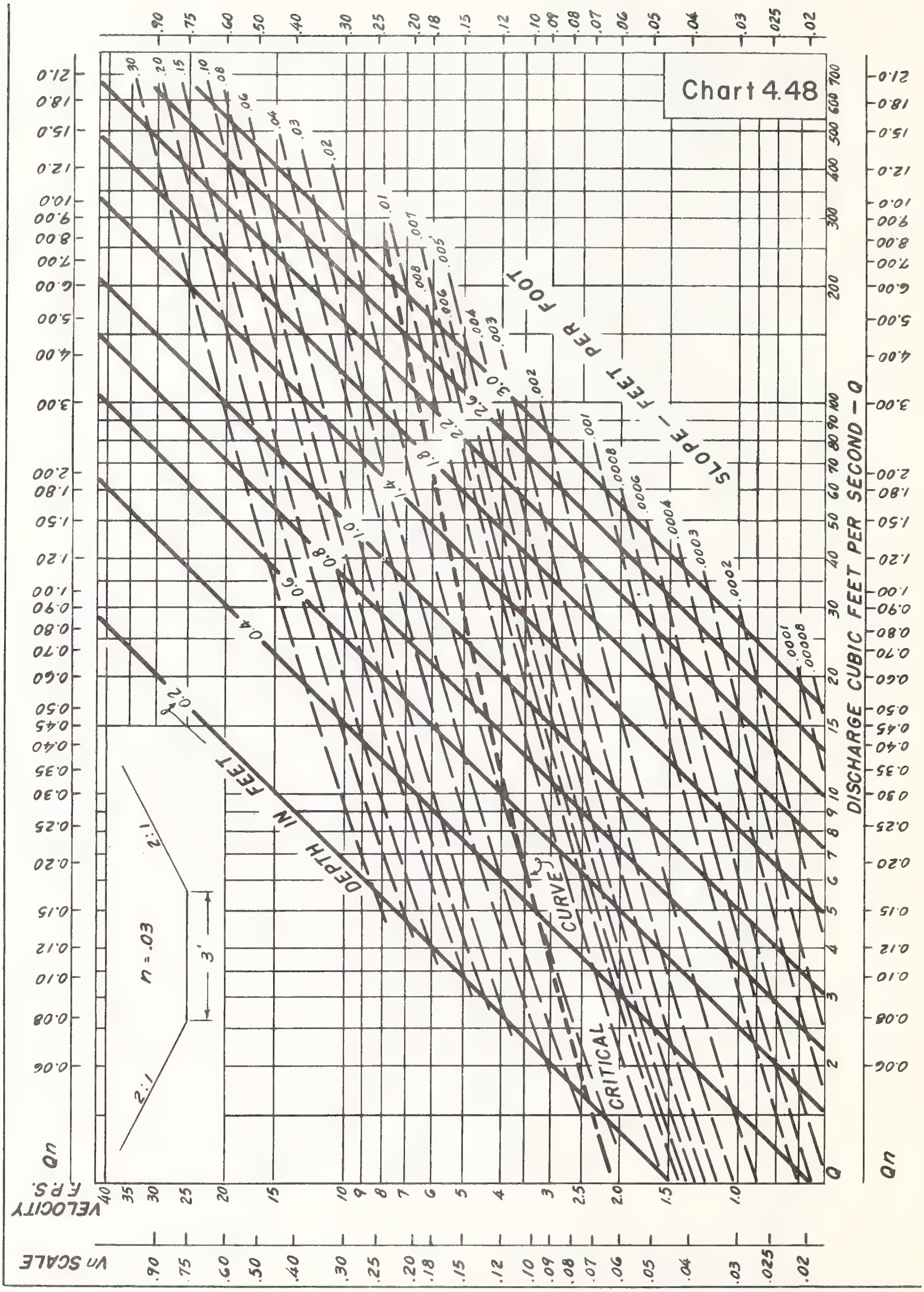
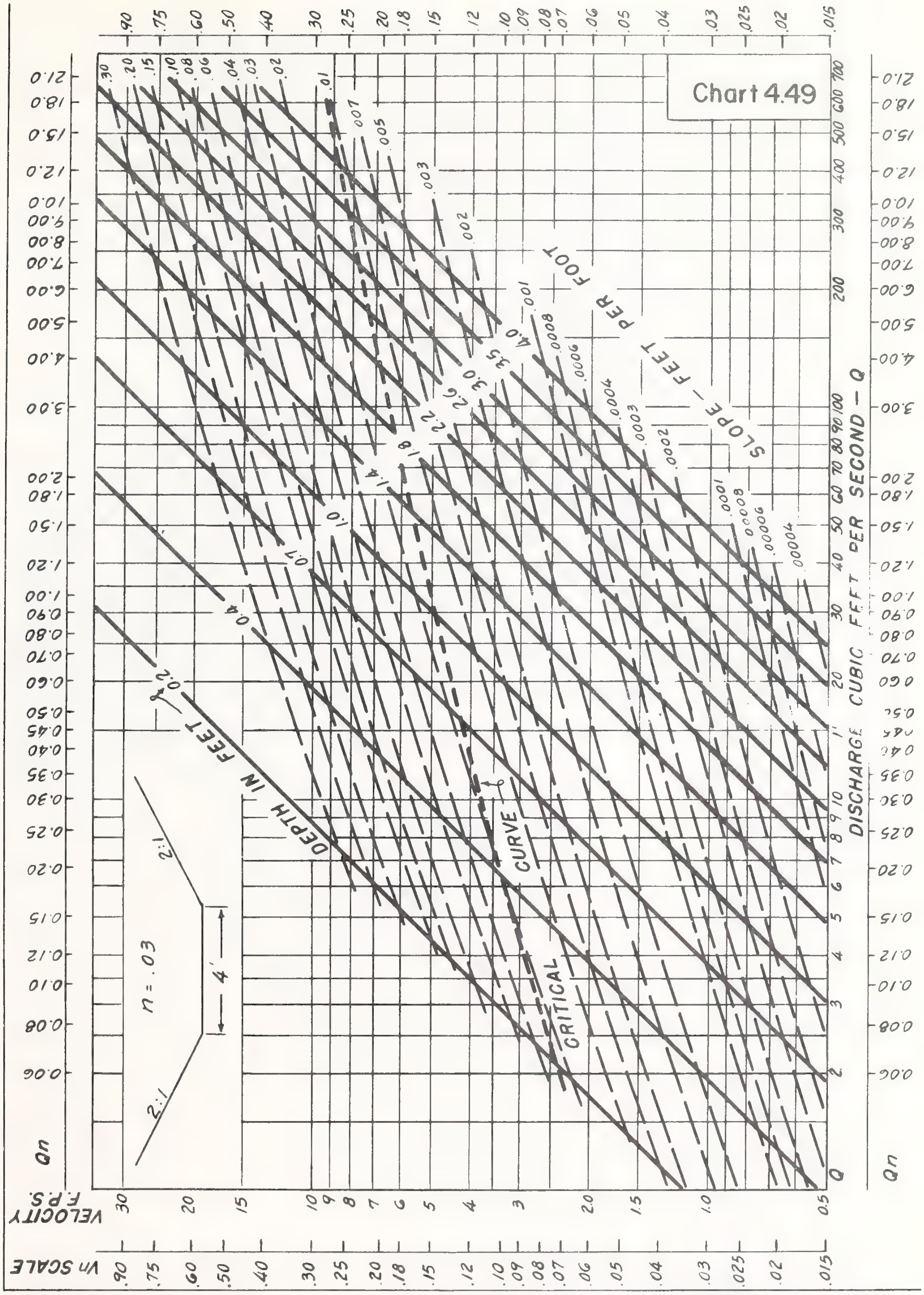


CHANNEL CHART
2:1 b = 3 FT.



CHANNEL CHART
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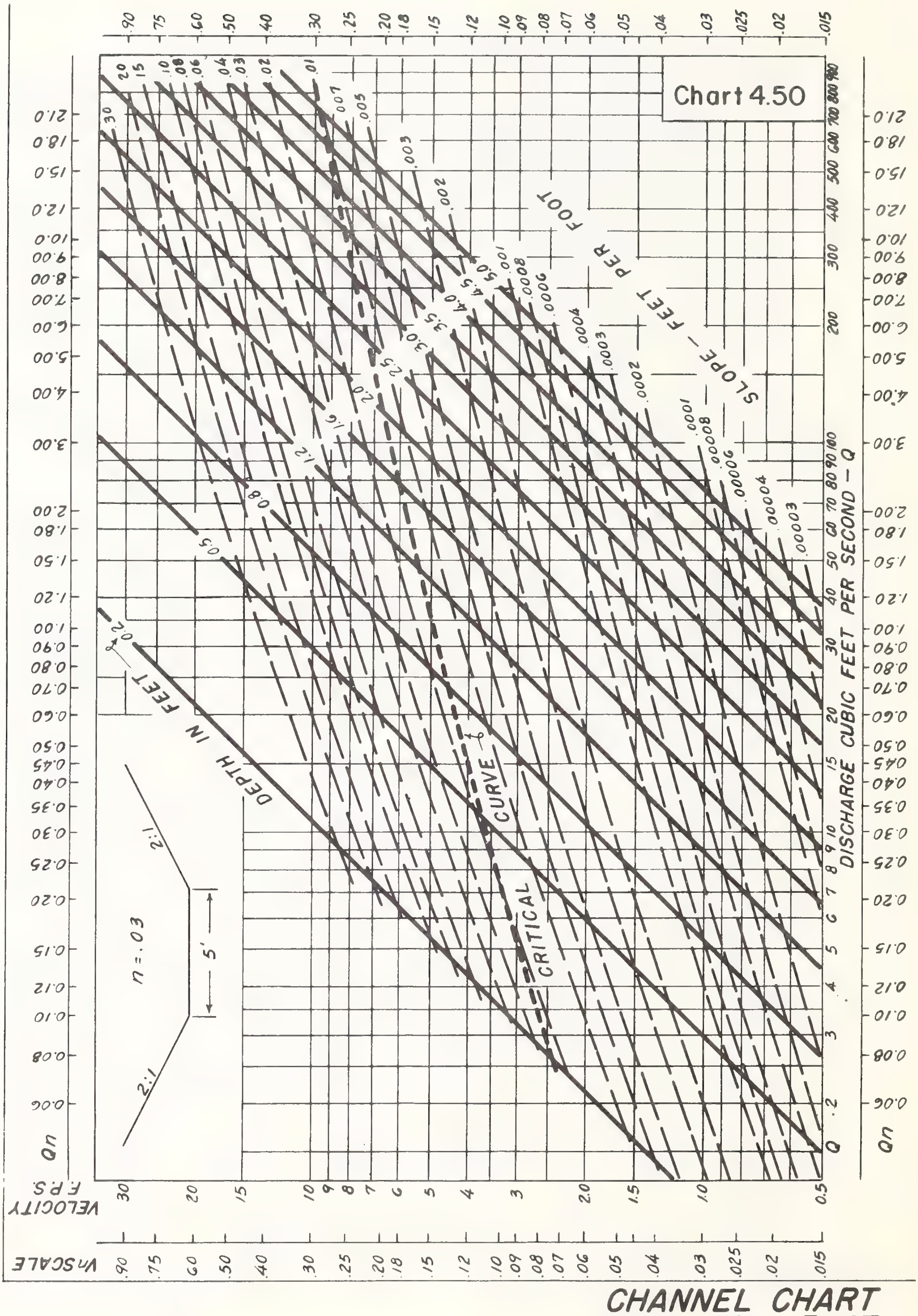
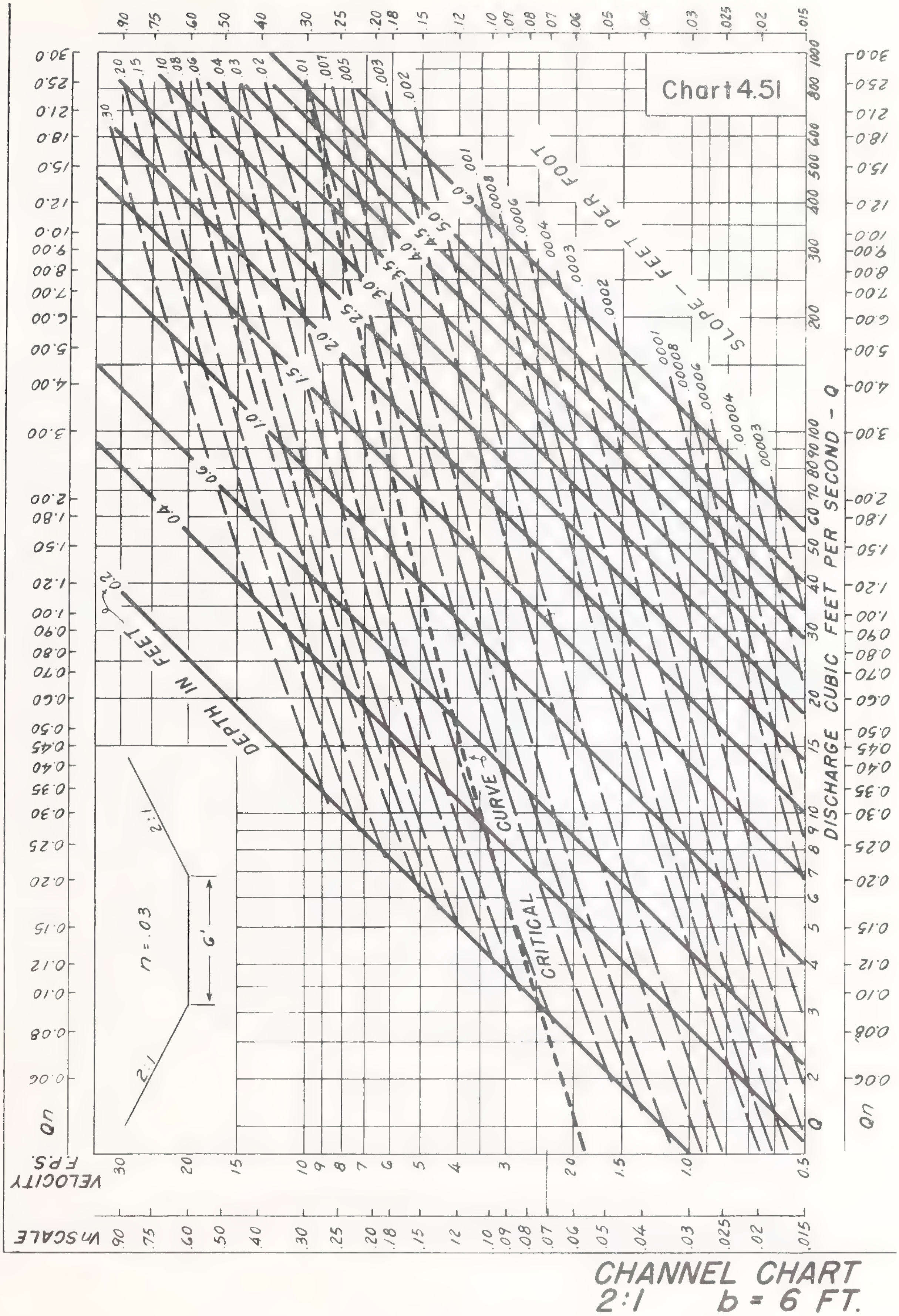


CHANNEL CHART
2:1 b = 5 FT.



CHANNEL CHART
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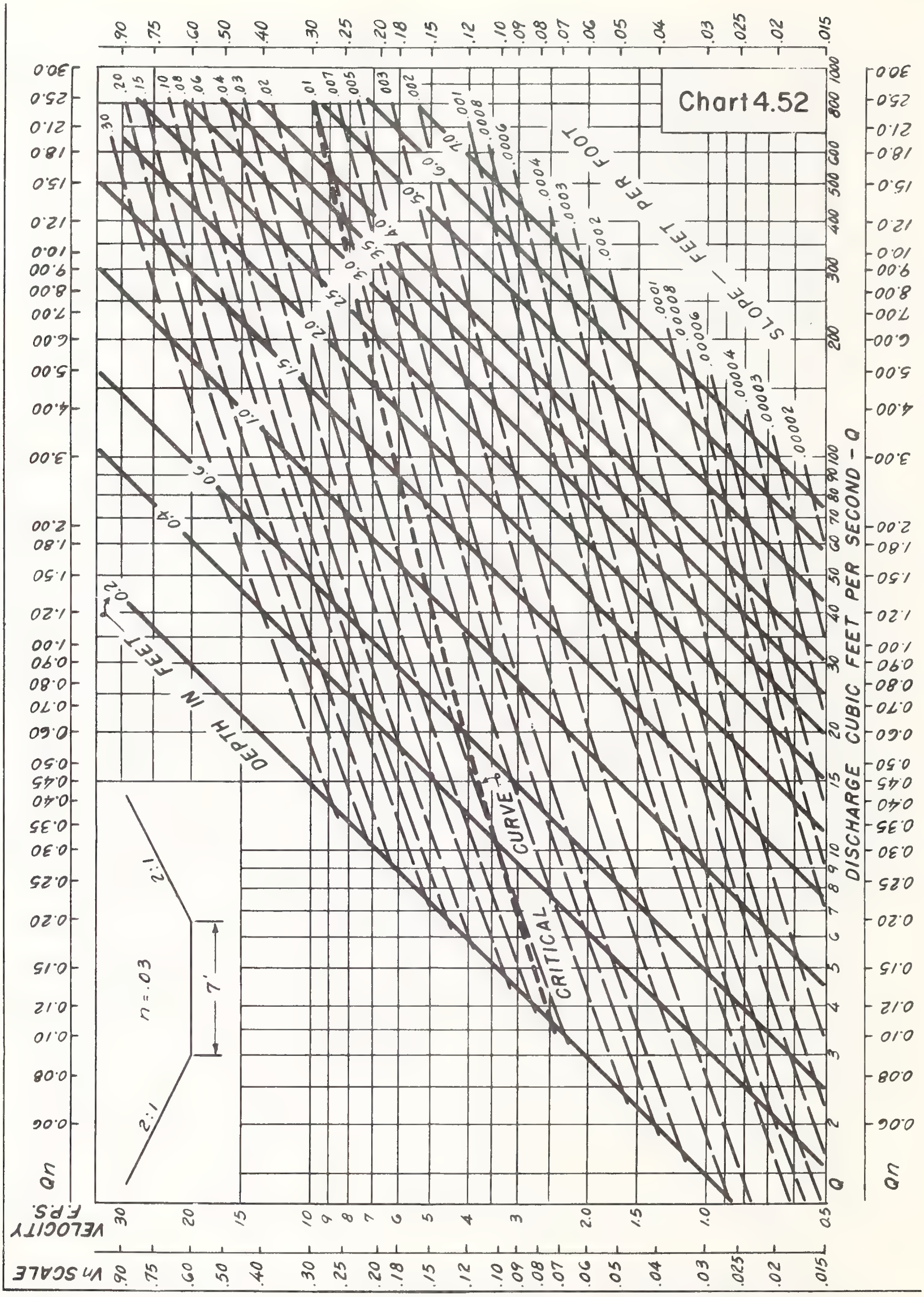
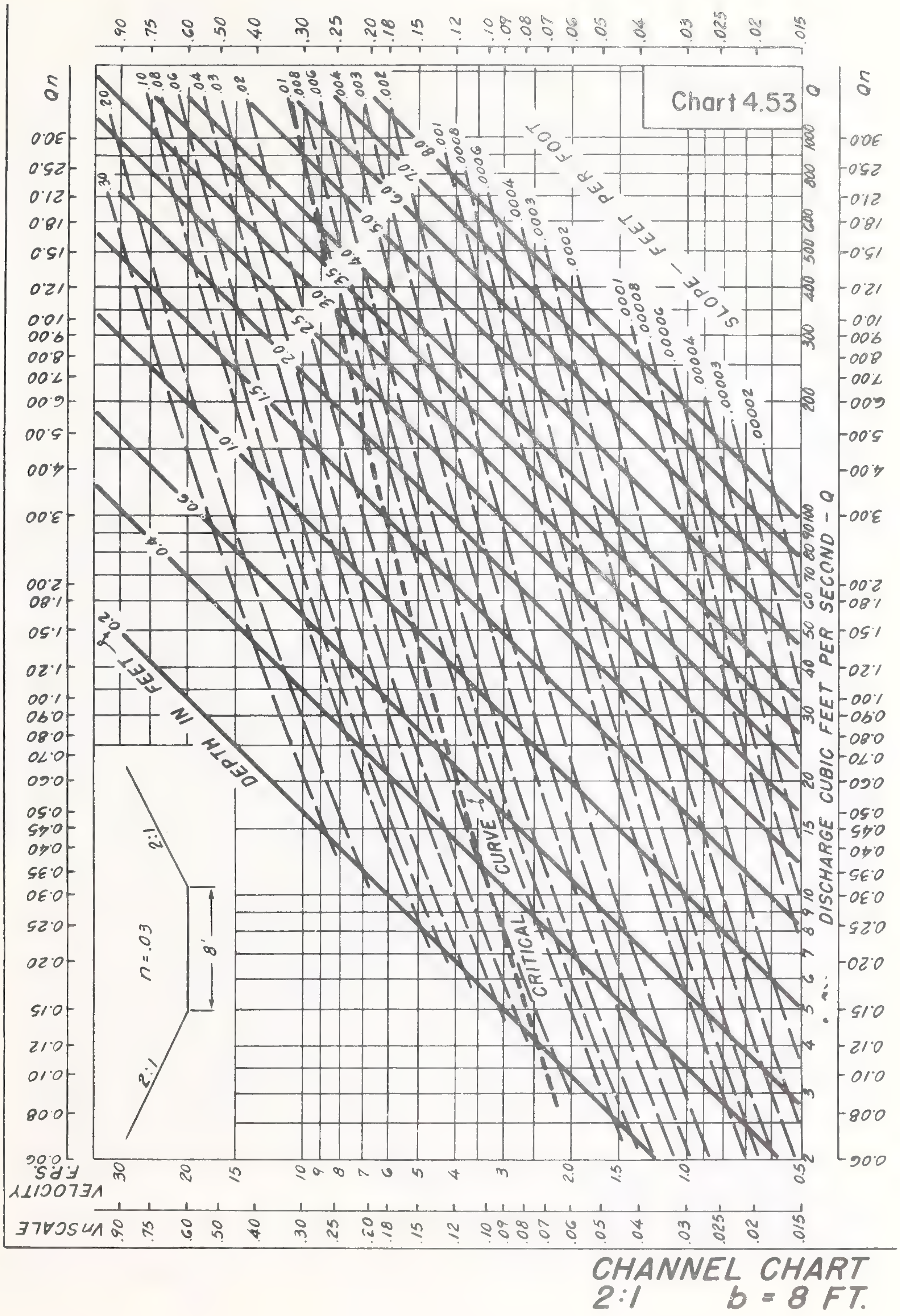


CHANNEL CHART
2:1 b = 7 FT.



CHANNEL CHART
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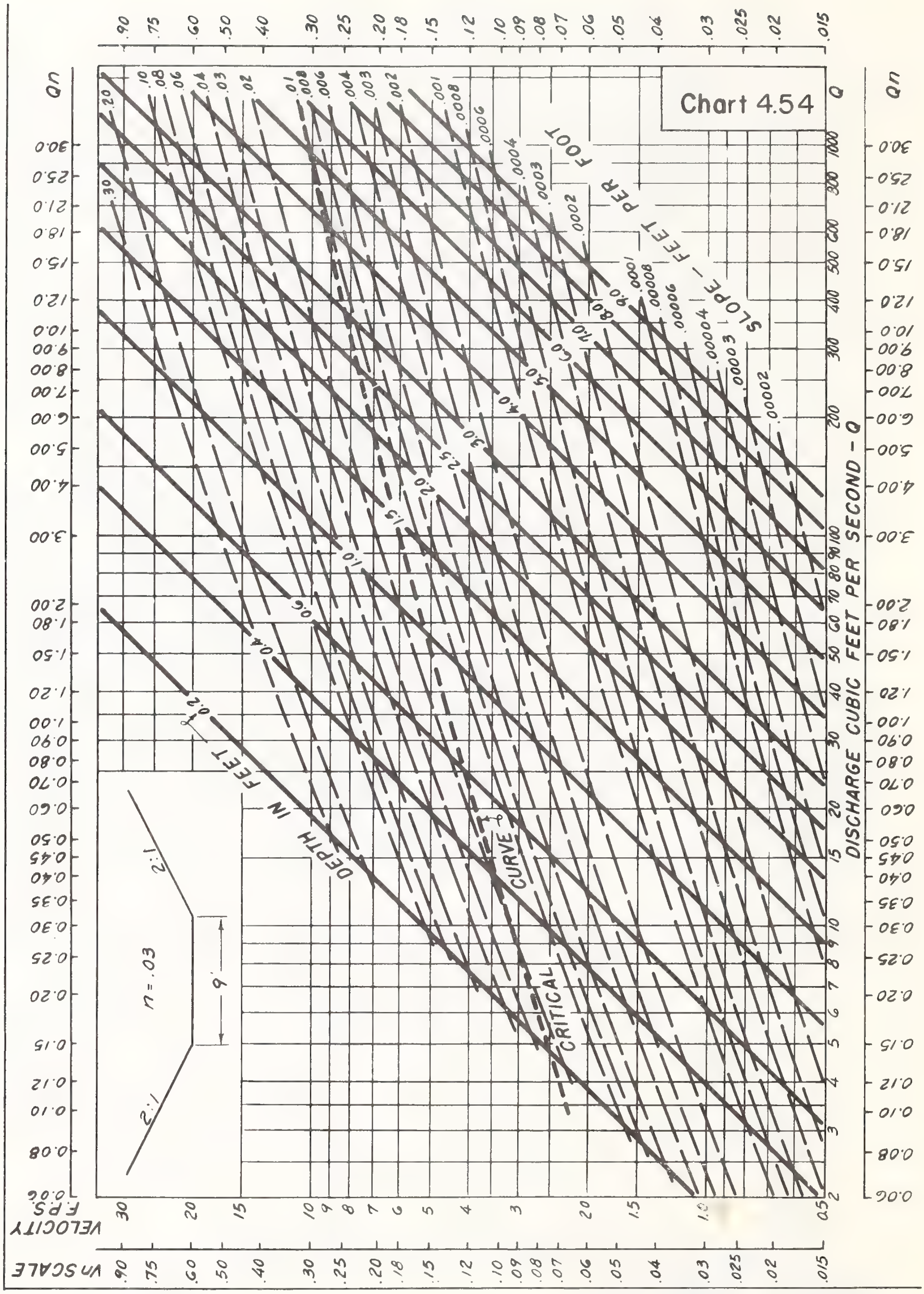

CHANNEL CHART
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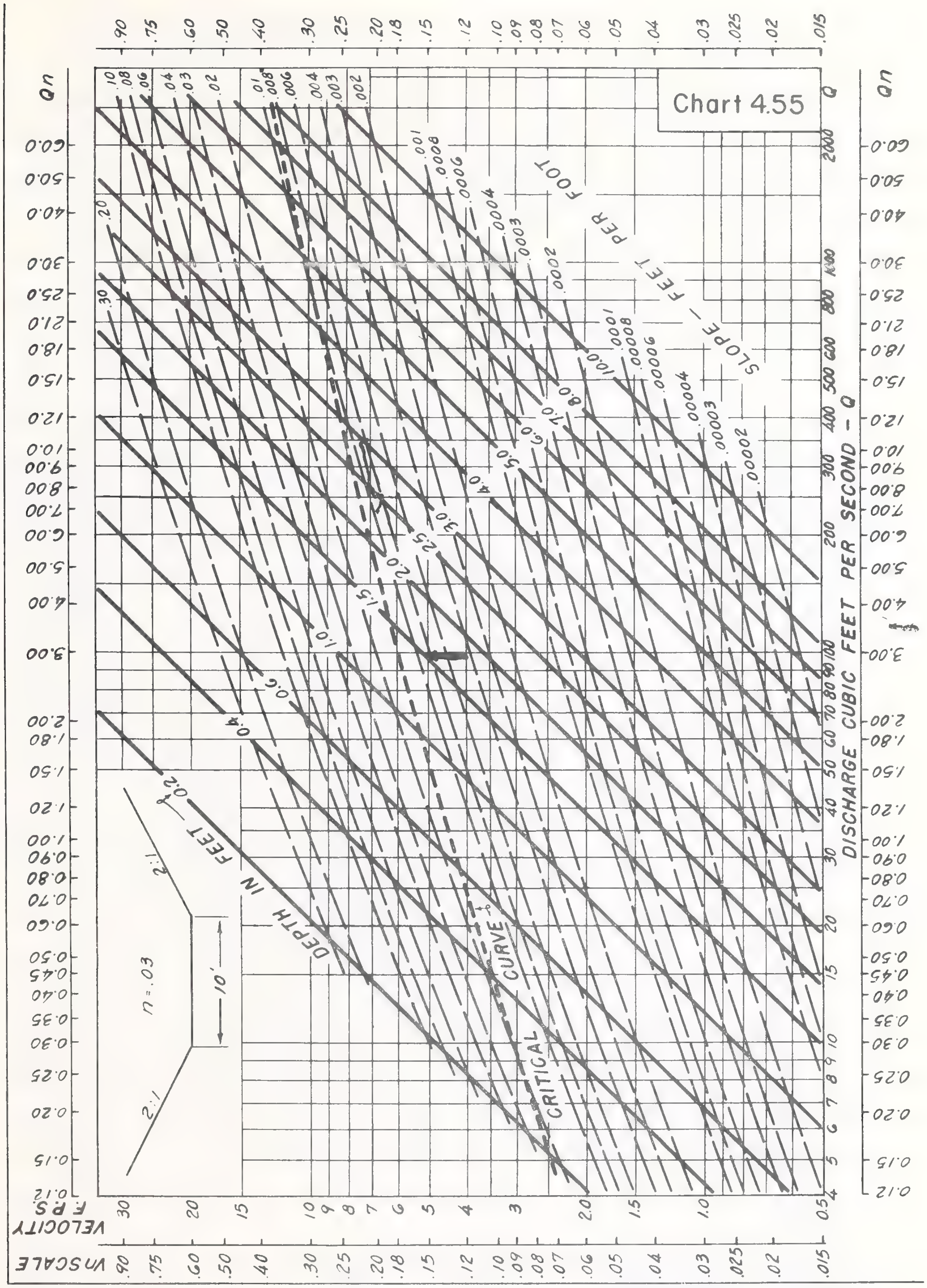
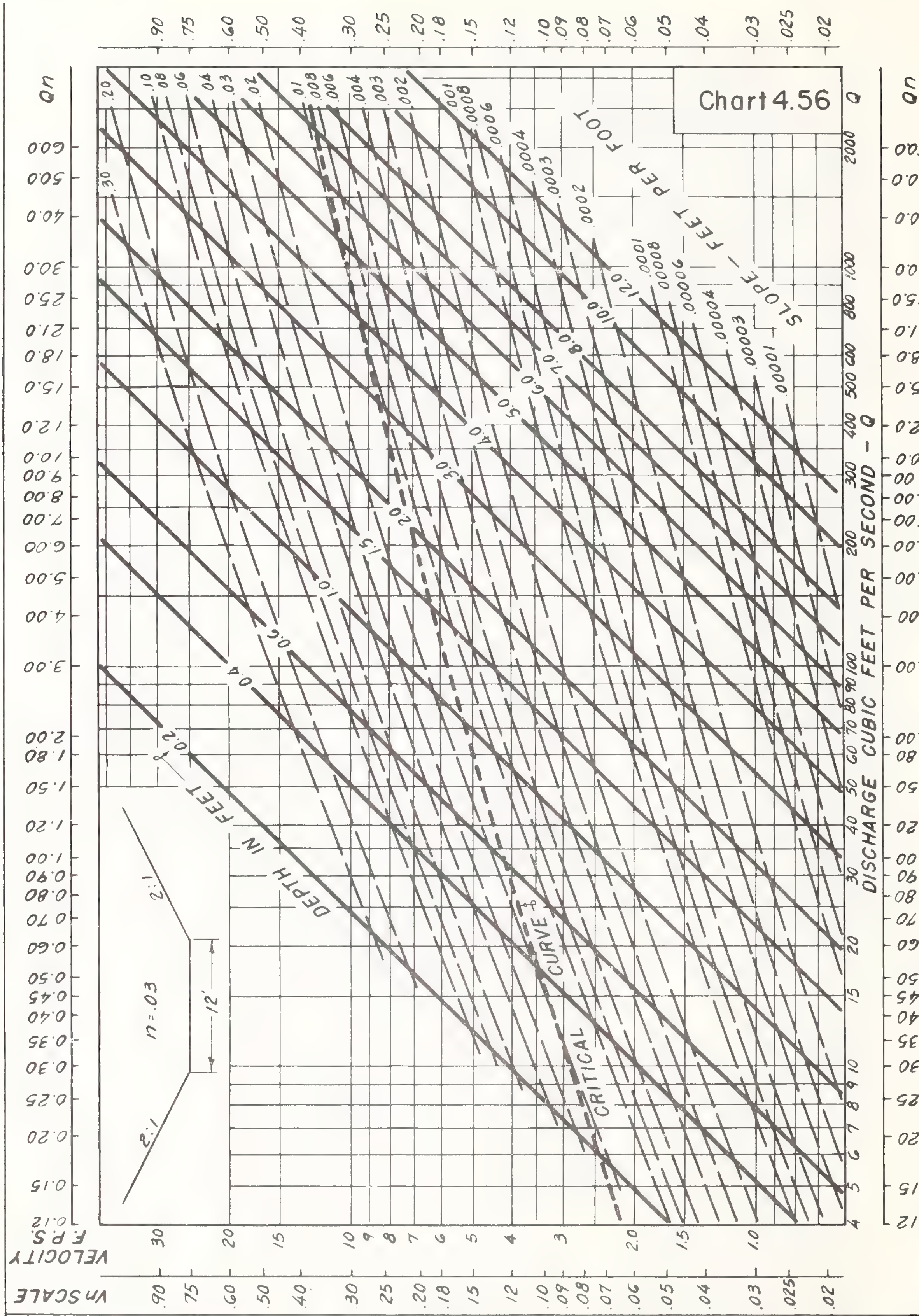


CHANNEL CHART
2:1 b = 10 FT.



CHANNEL CHART
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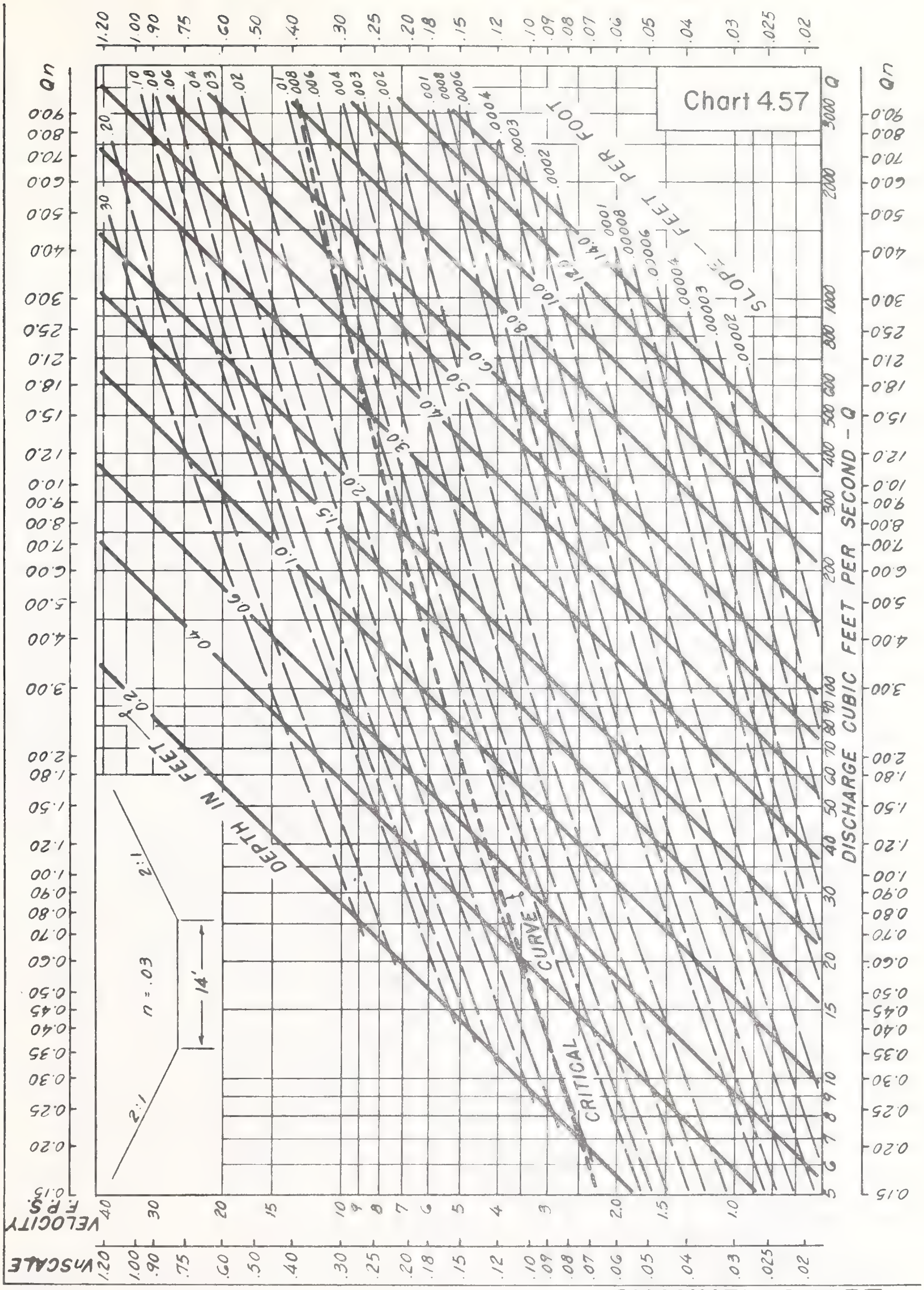

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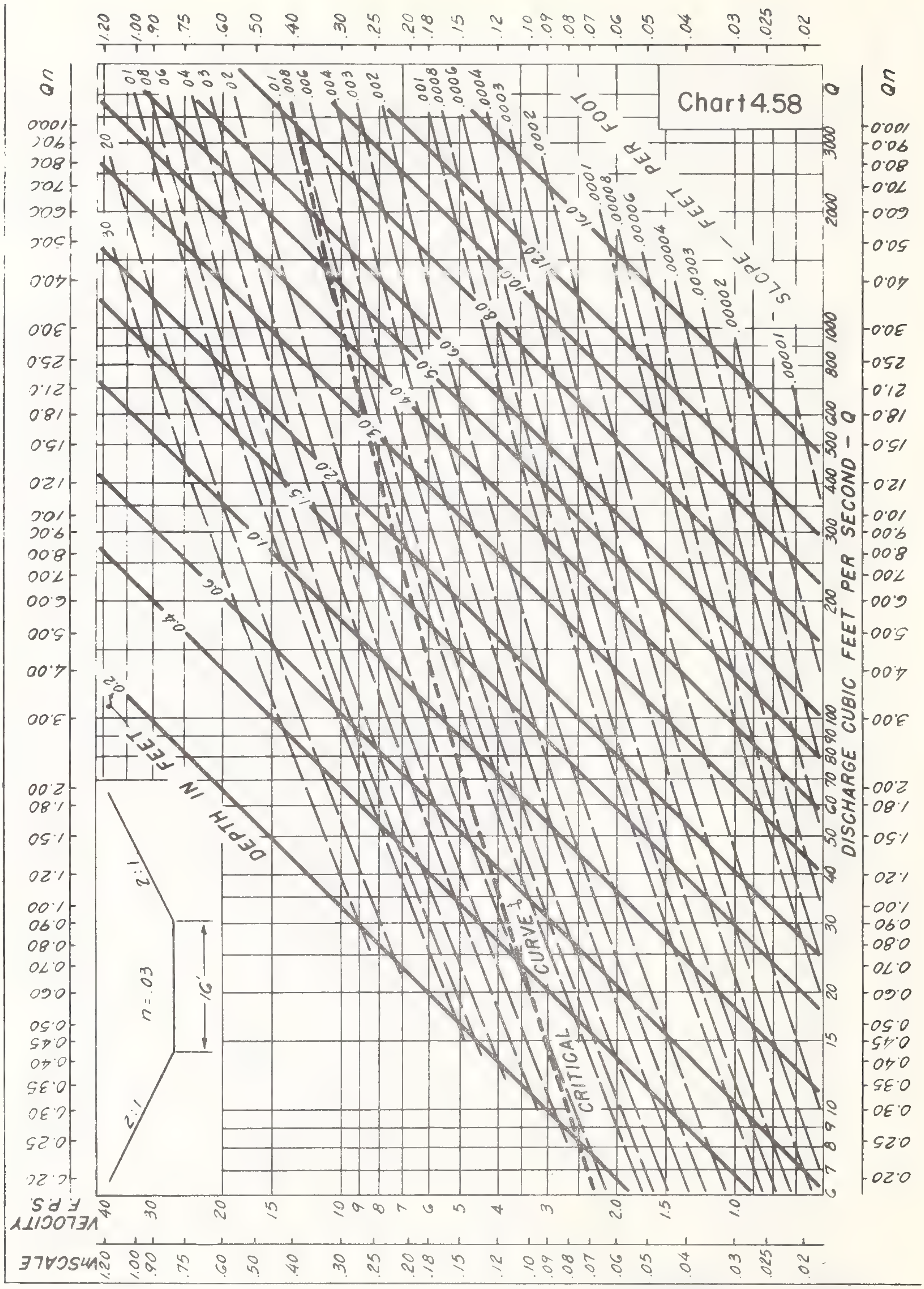

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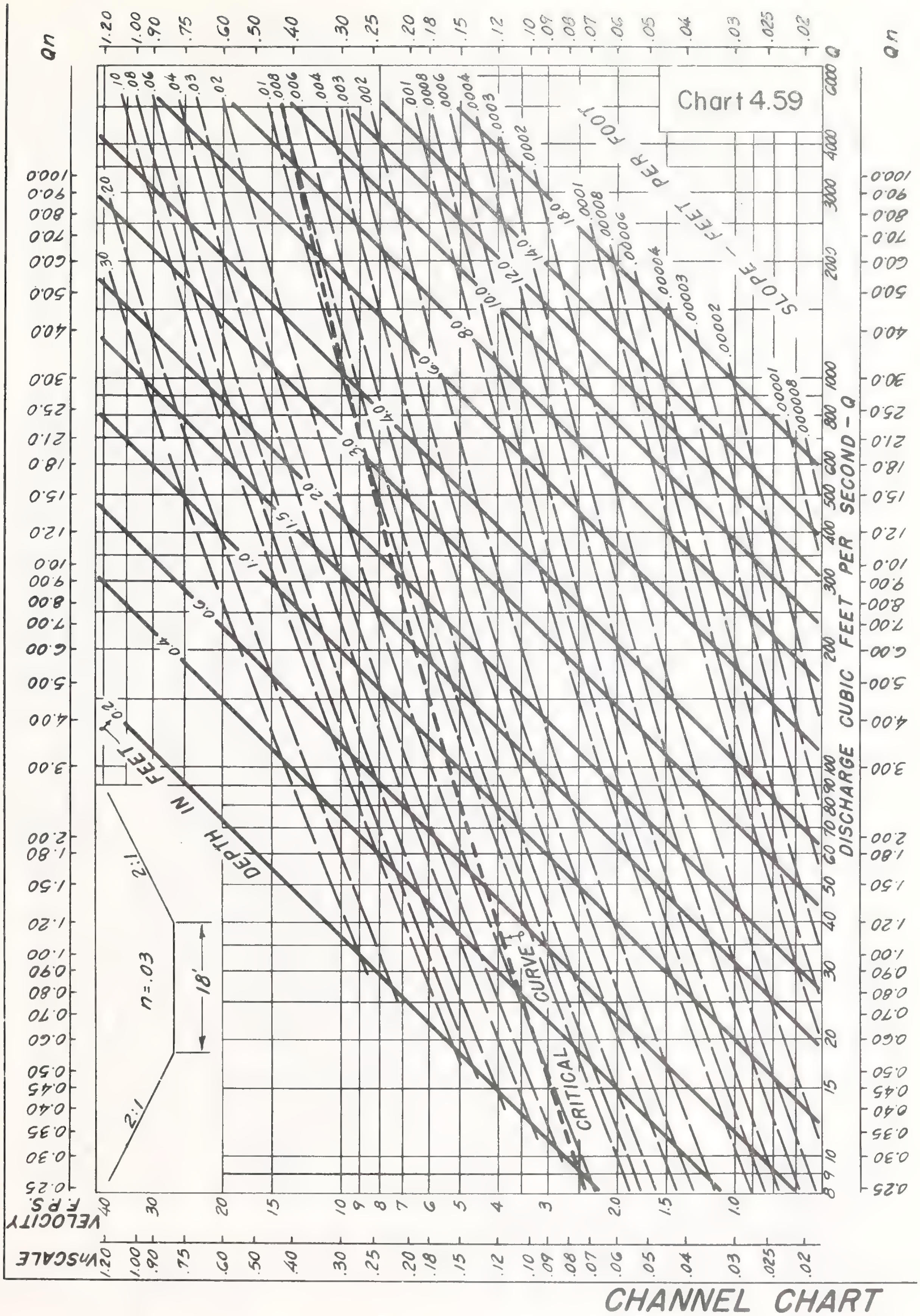
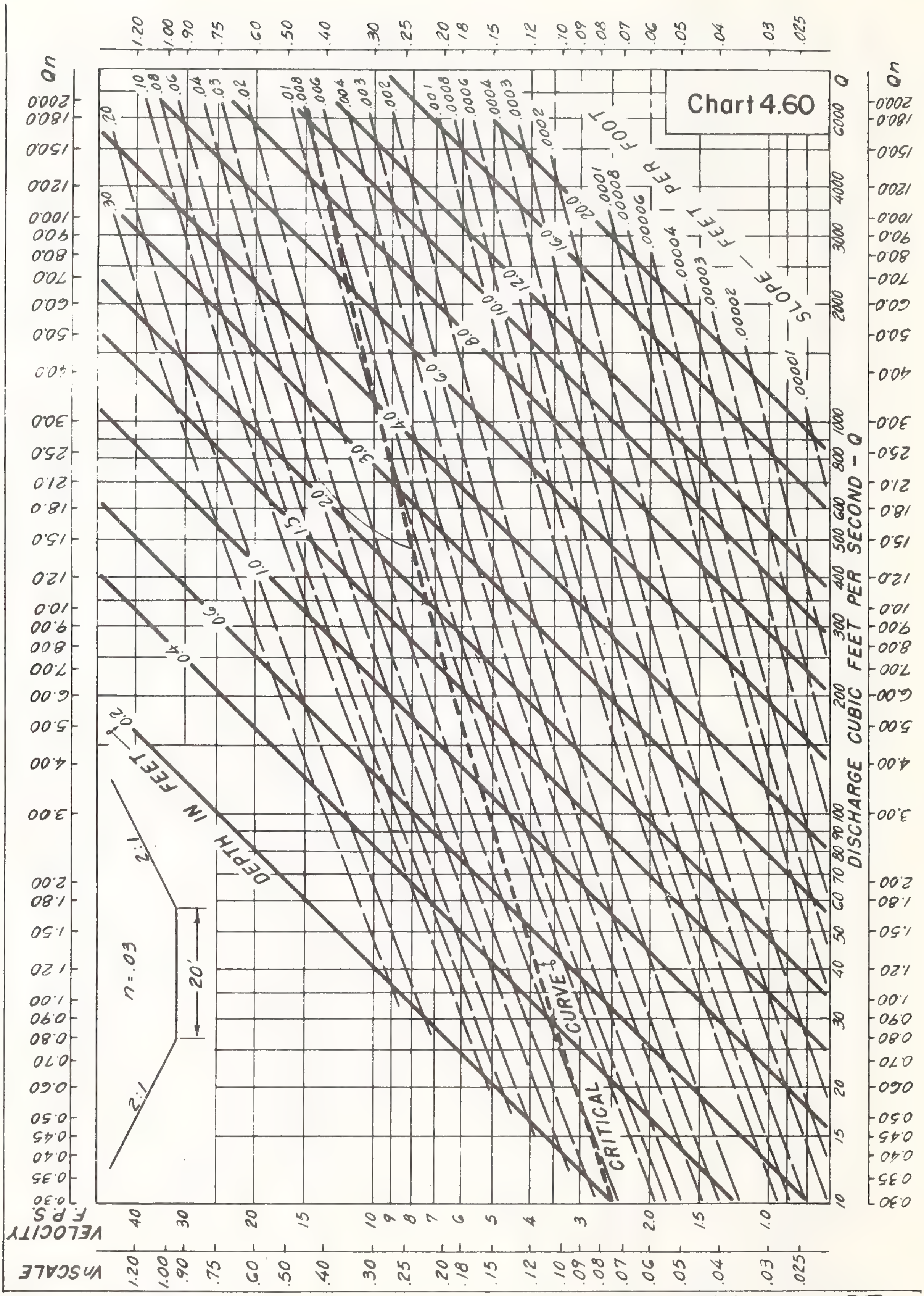
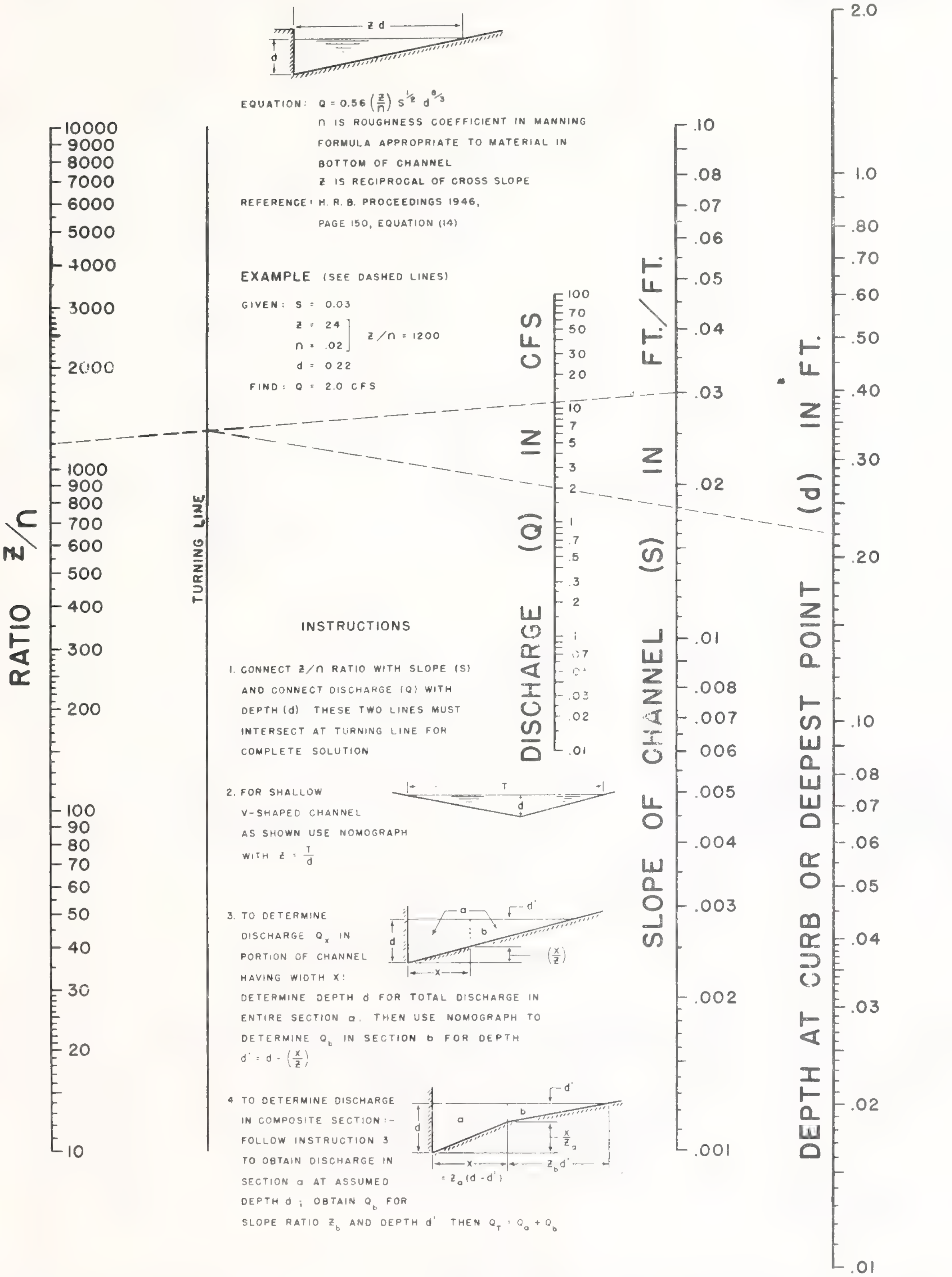


CHANNEL CHART
2:1 b = 18 FT.



CHANNEL CHART
2:1 b = 20 FT.



NOMOGRAPH FOR FLOW IN TRIANGULAR CHANNELS

4.63 GRASS LINED CHANNELS

<u>Description of Charts</u> - Charts 4.62 - 4.66 are designed for use in the direct solution of the Manning equation for various channel section lined with grass. Charts 4.62 - 4.65 are for trapezoidal cross section channels, in each case with a 4-foot bottom width, but with side slopes, respectively, of 2:1, 4:1, 6:1, and 8:1. Chart 4.66 is for a triangular cross section channel with a side slope of 10:1.

The charts are similar in appearance and use to those for trapezoidal cross sections (Charts 4.47 - 4.60). However, their construction is much more difficult because the roughness coefficient n varies with the type and height of grass and with the velocity and depth of flow. The effect of velocity and depth of flow on n may be evaluated by the product of velocity and hydraulic radius, VR. The variation of Manning's n with the retardance and the product VR is shown in Figure 4.48 in which four retardance curves are shown. The retardance varies with the height of the grass and the condition of the stand, as indicated in Table 4.33. Both of these factors depend upon the type of grass, planting conditions, and maintenance practices.

Each of Charts 4.62 - 4.66 has two graphs, the upper graph being for retardance D and the lower graph for retardance C. For grasses commonly used in roadway drainage channels, such as Bermudagrass, Kentucky bluegrass, orchard-grass, redtop, Italian ryegrass, and buffalograss, the retardance may be selected from Table 4.33A.

The charts are plotted with discharge, in cubic feet per second, as the abscissa and slope, in feet per foot, as the ordinate. Both scales are logarithmic. Superimposed on the logarithmic grid are lines for velocity, in feet per second, and lines for depth, in feet. A dashed line shows the position of critical flow.

General Instructions for Use of Charts 4.62 - 4.66 - Charts 4.62 - 4.66 provide a solution of the Manning equation for flow in open grassed channels of uniform slope and cross section, provided the flow is not affected by backwater and the channel has a length sufficient to establish uniform flow. The charts provide accuracy sufficient for design of highway drainage channels of fairly uniform cross section and slope. Rounding of the intersection of the side slopes with the bottom of the channel does not appreciably affect the channel capacity.

The actual design of grass lined channels should be preformed using the procedures of Section 4.82. This section and these charts are provided so the designer can check the capacity of existing channels.

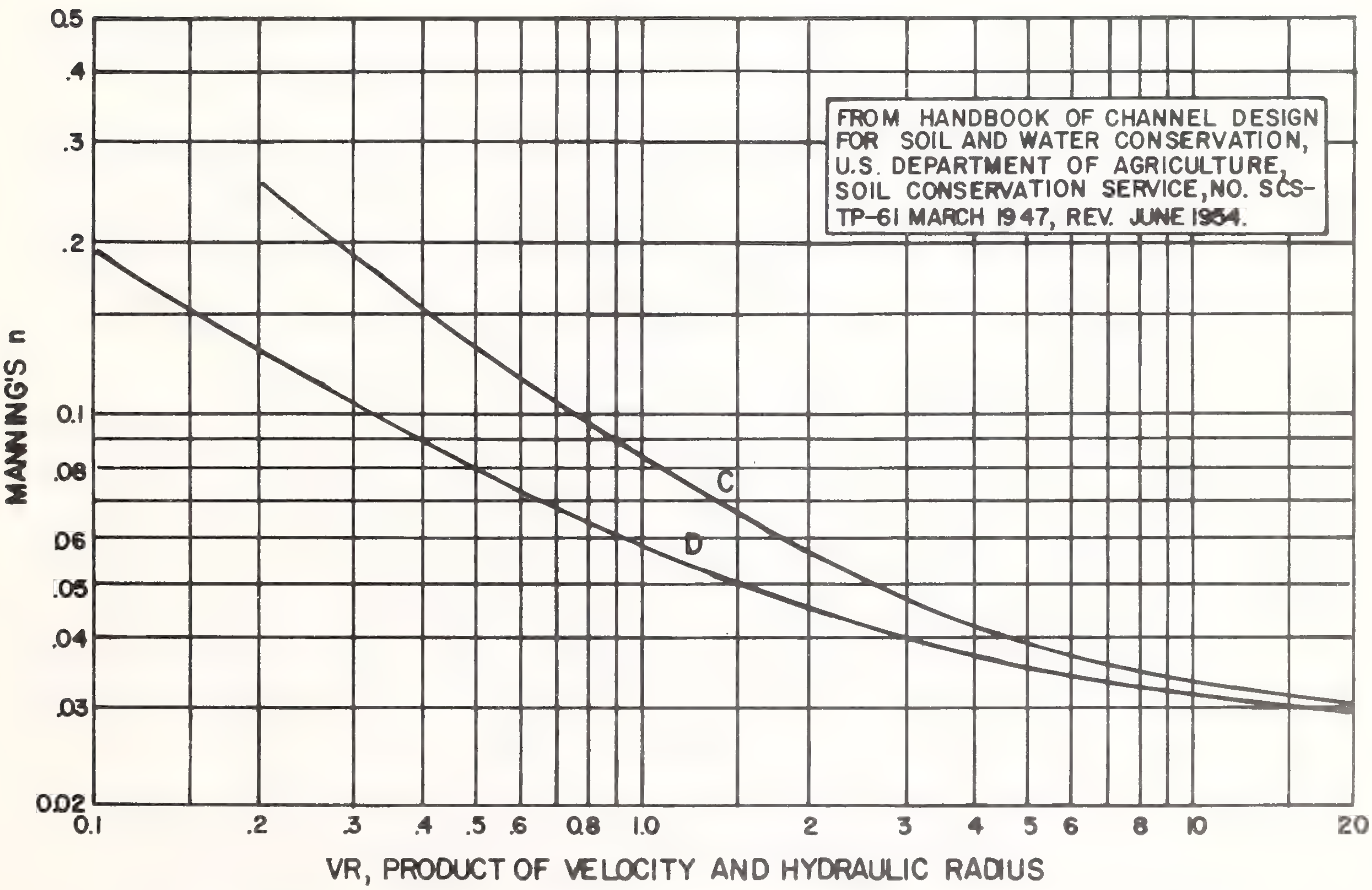


Figure 4.48-Degrees of vegetal retardance for which the Manning n has been determined.

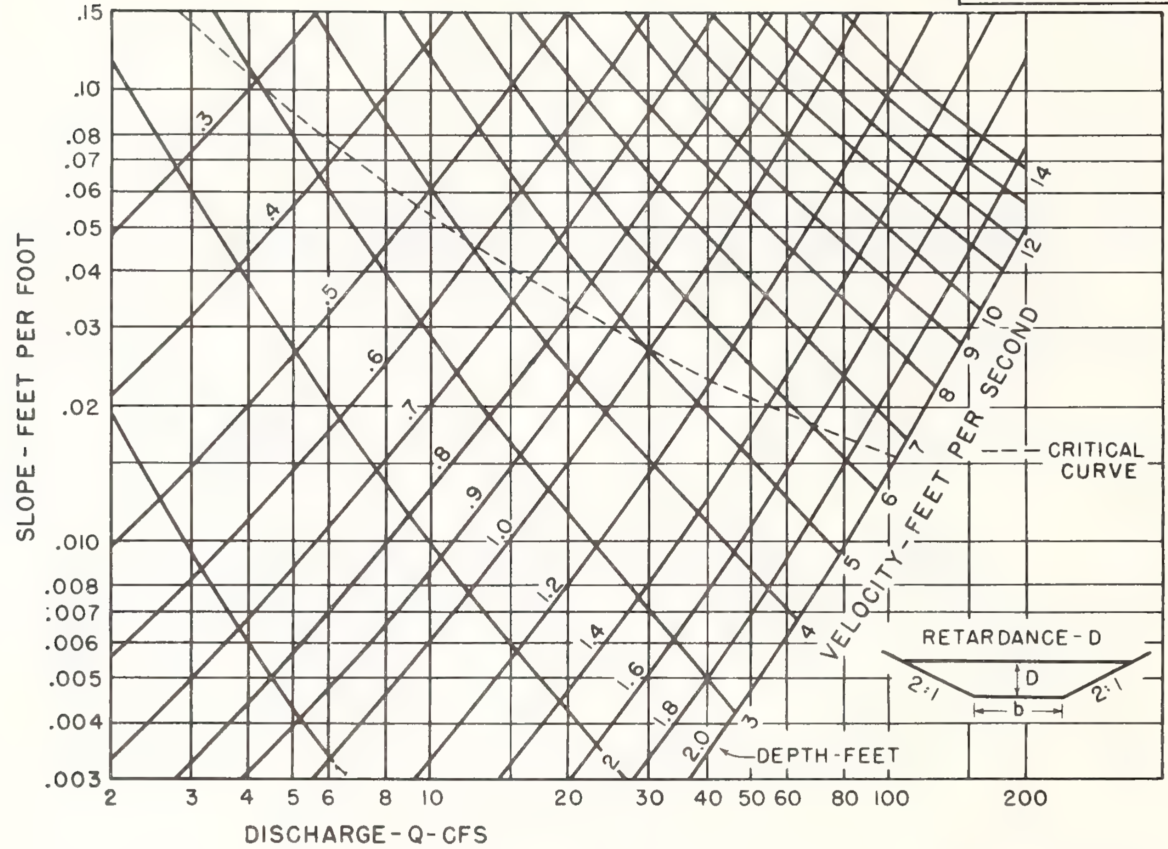
Table 4.33 - Guide to selection of retardance curve

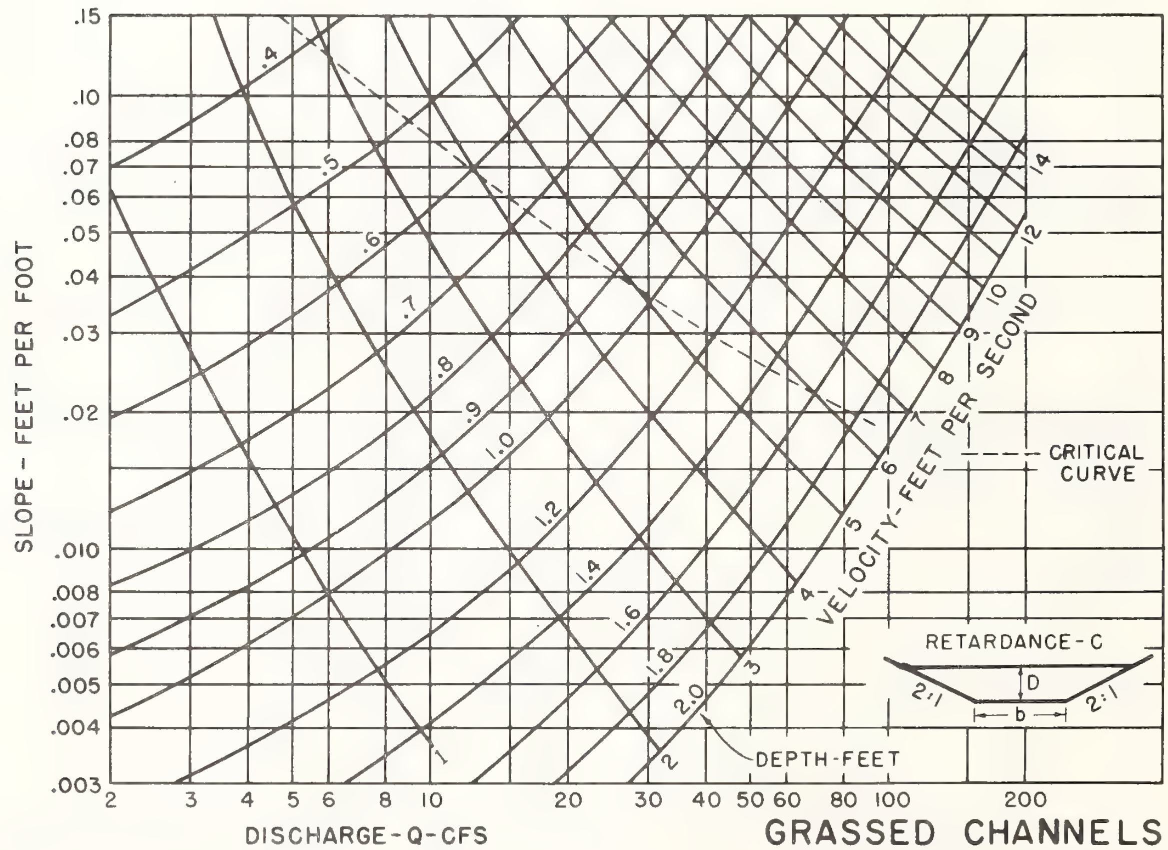
Average length of vegetation	Retardance curve for-	
	Good Stand	Fair Stand
6-10 inches..	C.......	D.
2-6 inches...	D.......	D.

Table 4.33A-Permissible velocities for channels lined with uniform stands of various grass covers, well maintained [1] [2]

Cover	Slope range	Permissible velocity on— Erosion resistant soils	Permissible velocity on— Easily eroded soils
	Percent	F.p.s.	F.p.s.
Bermudagrass	0-5	8	6
	5-10	7	5
	Over 10	6	4
Buffalograss, Kentucky bluegrass, Smooth brome, Blue grama	0-5	7	5
	5-10	6	4
	Over 10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
Lespedeza sericea, Weeping lovegrass, Yellow bluestem, Kudzu, Alfalfa, Crabgrass	0-5	3.5	2.5
Common lespedeza[3], Sudangrass[3]	[4]0-5	3.5	2.5

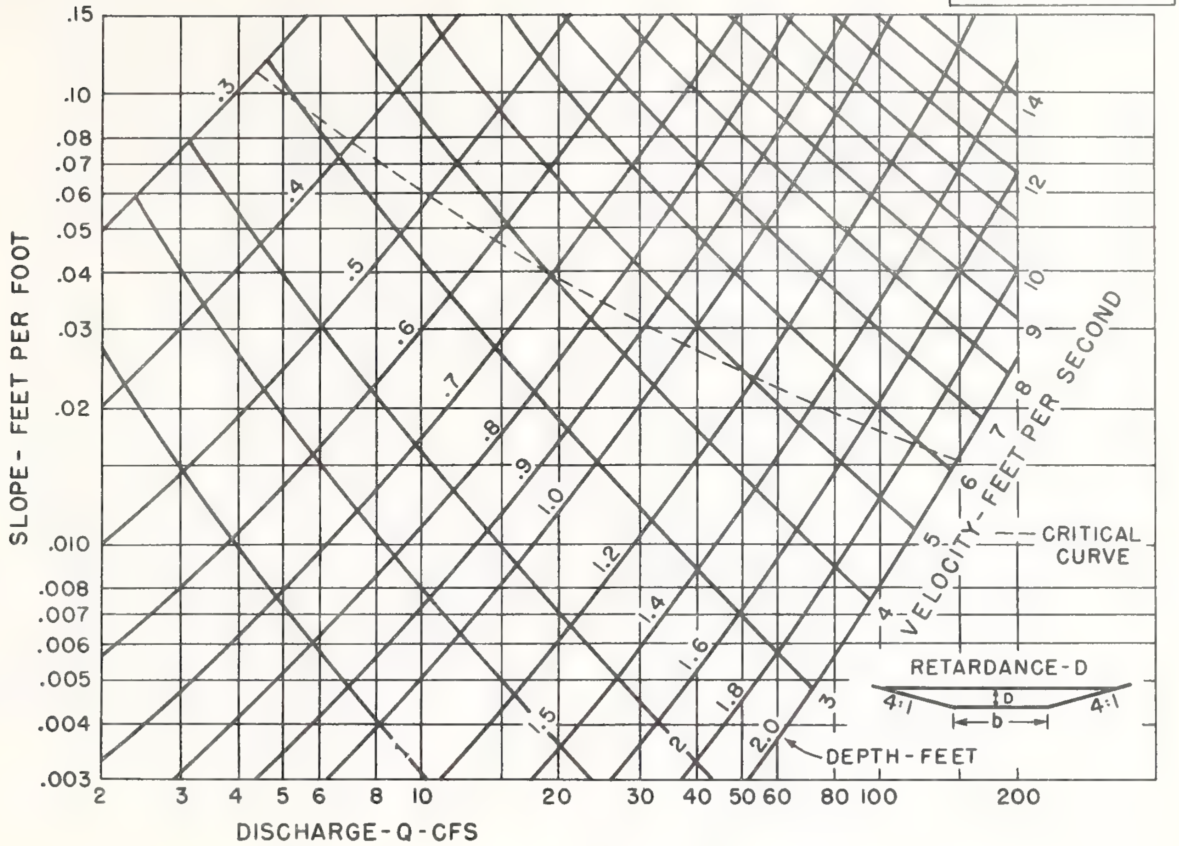
[1]From *Handbook of Channel Design for Soil and Water Conservation.*
[2]Use velocities over 5 f.p.s. only where good covers and proper maintenance can be obtained.
[3]Annuals, used on mild slopes or as temporary protection until permanent covers are established.
[4]Use on slopes steeper than 5 percent is not recommended.

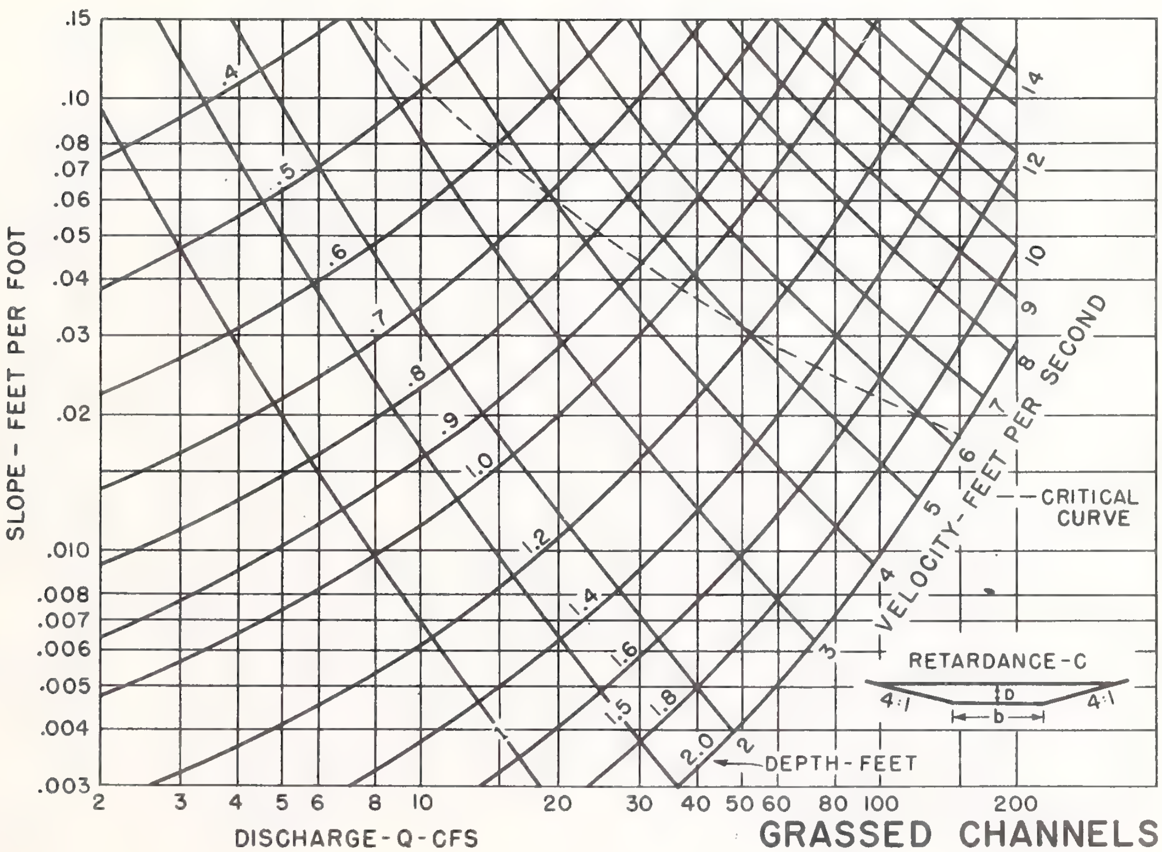




GRASSED CHANNELS

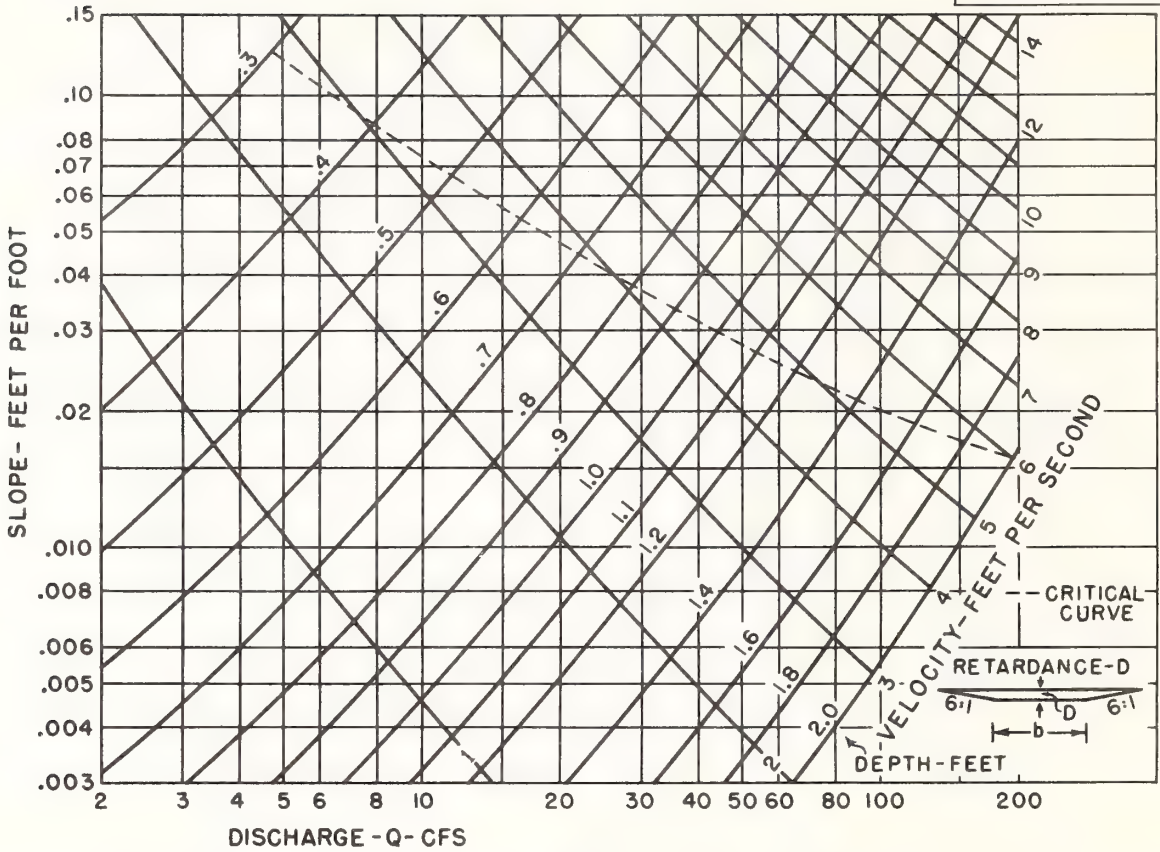
2:1 b = 4 ft.

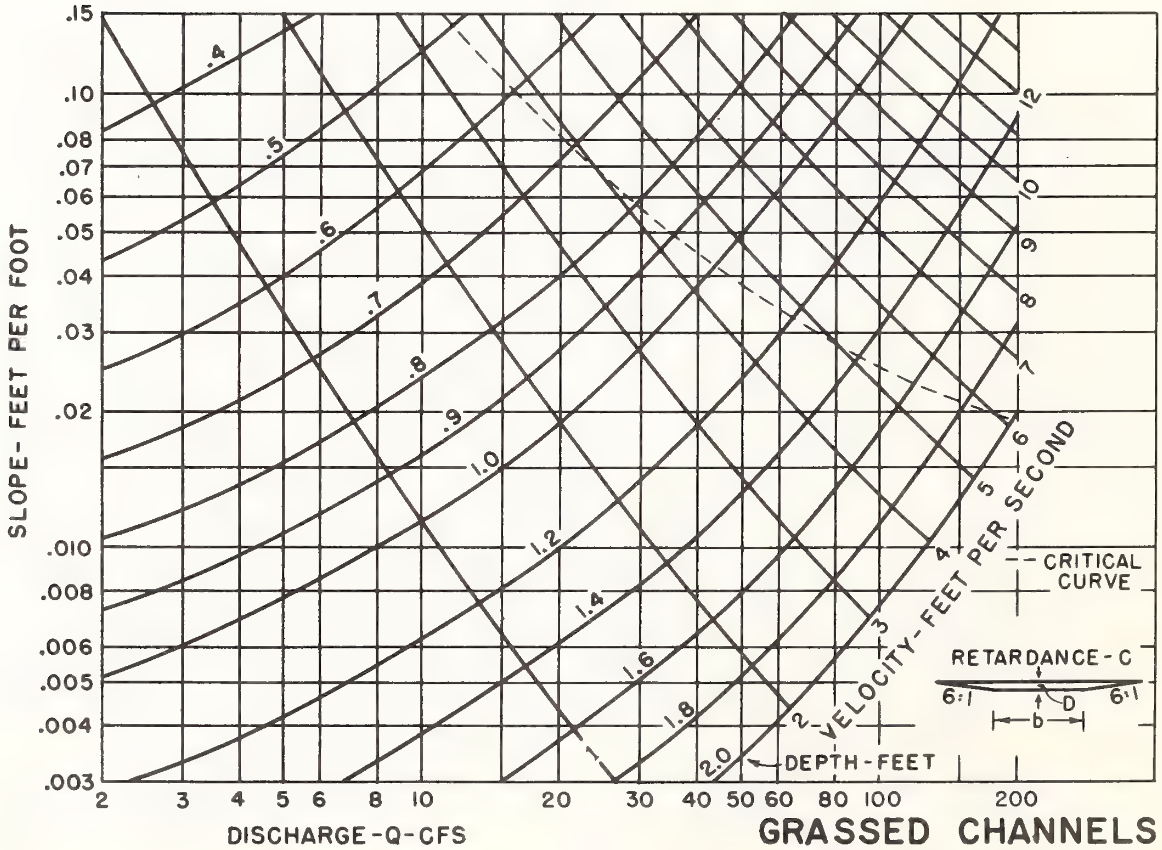




GRASSED CHANNELS

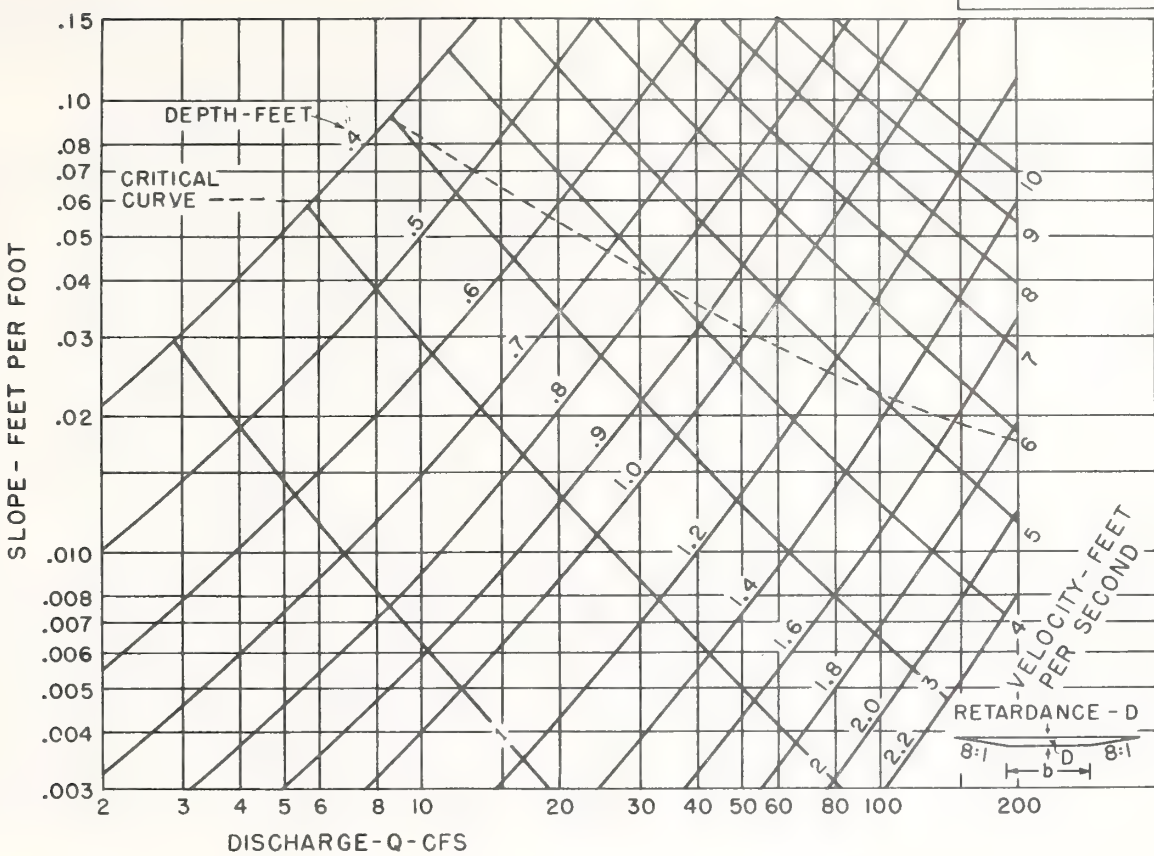
4:1 b = 4 ft.

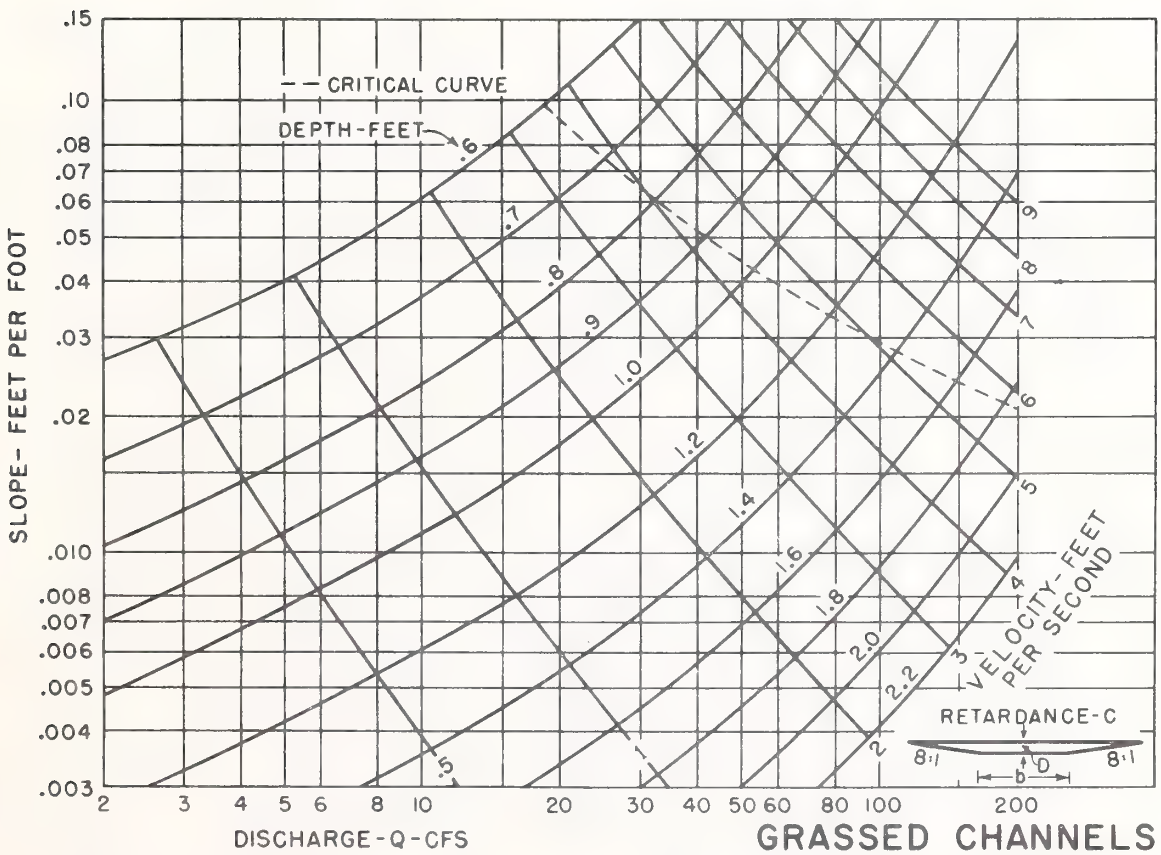




GRASSED CHANNELS
6:1 b = 4 ft.

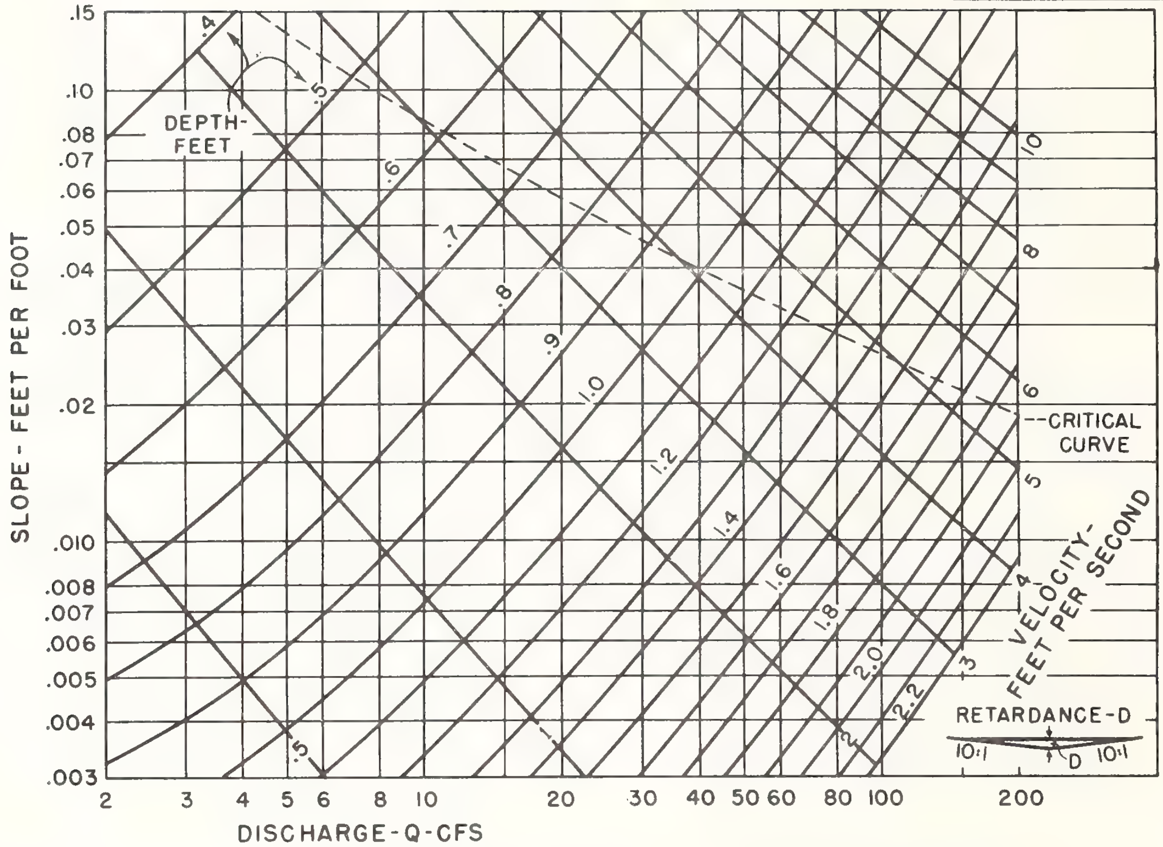
Chart 4.65

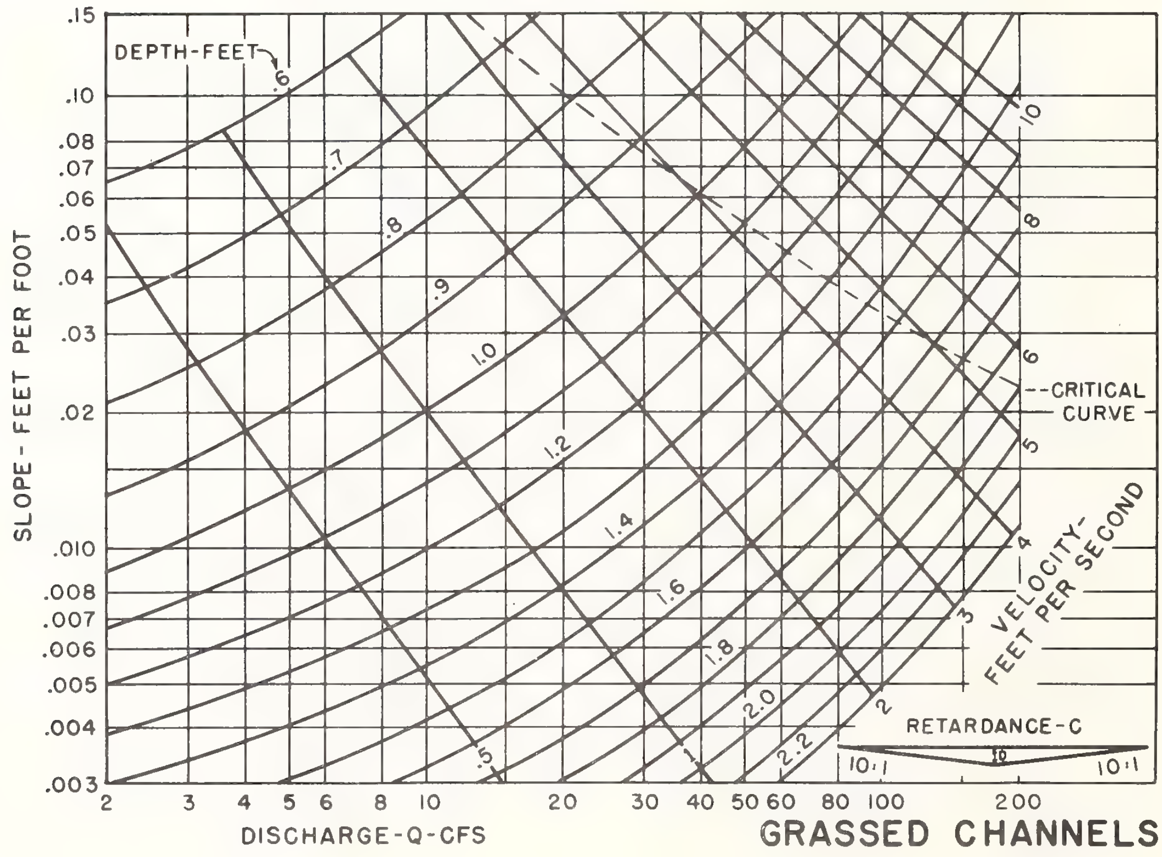




GRASSED CHANNELS

8:1 b = 4 ft.





GRASSED CHANNELS
10:1

4.64 CIRCULAR PIPE CHANNELS

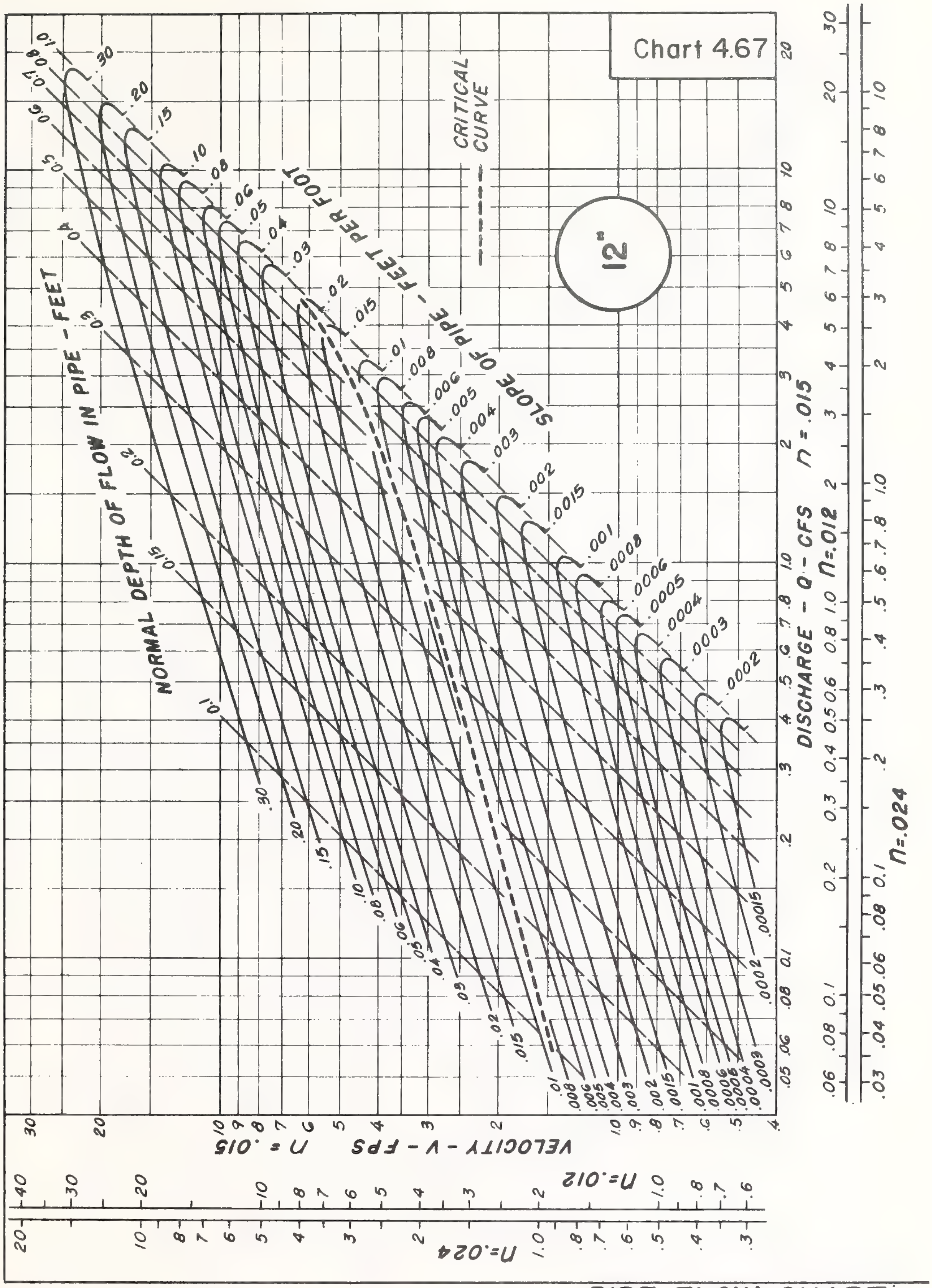
<u>Description of Charts</u> - Charts 4.67 - 4.92 are designed for use in the solution of the Manning equation for circular pipe channels which have sufficient length, on constant slope, to establish uniform flow at normal depth without backwater or pressure head. It is important to recognize that they are not suitable for use in connection with most types of culvert flow, since culvert flow is seldom uniform.

The Charts are of two types. Charts 4.67 - 4.83 are similar to the open-channel charts of Section 4.62. Separate charts are provided for pipe diameters of 12-36 inches, by 3-inch increments; for diameters of 42-72 inches, by 6-inch increments; and for diameters of 84 and 96 inches. The charts are prepared for an n of 0.015, with auxiliary scales for n = 0.012 and 0.024. The charts have an abscissa scale of discharge, in cubic feet per second, and an ordinate scale of velocity, in feet per second. Both scales are logarithmic. Superimposed on the logarithmic grid are steeply inclined lines representing depth (in feet), and slightly inclined lines representing channel slope (in feet per foot). A heavy dashed line on each chart shows the position of critical flow.

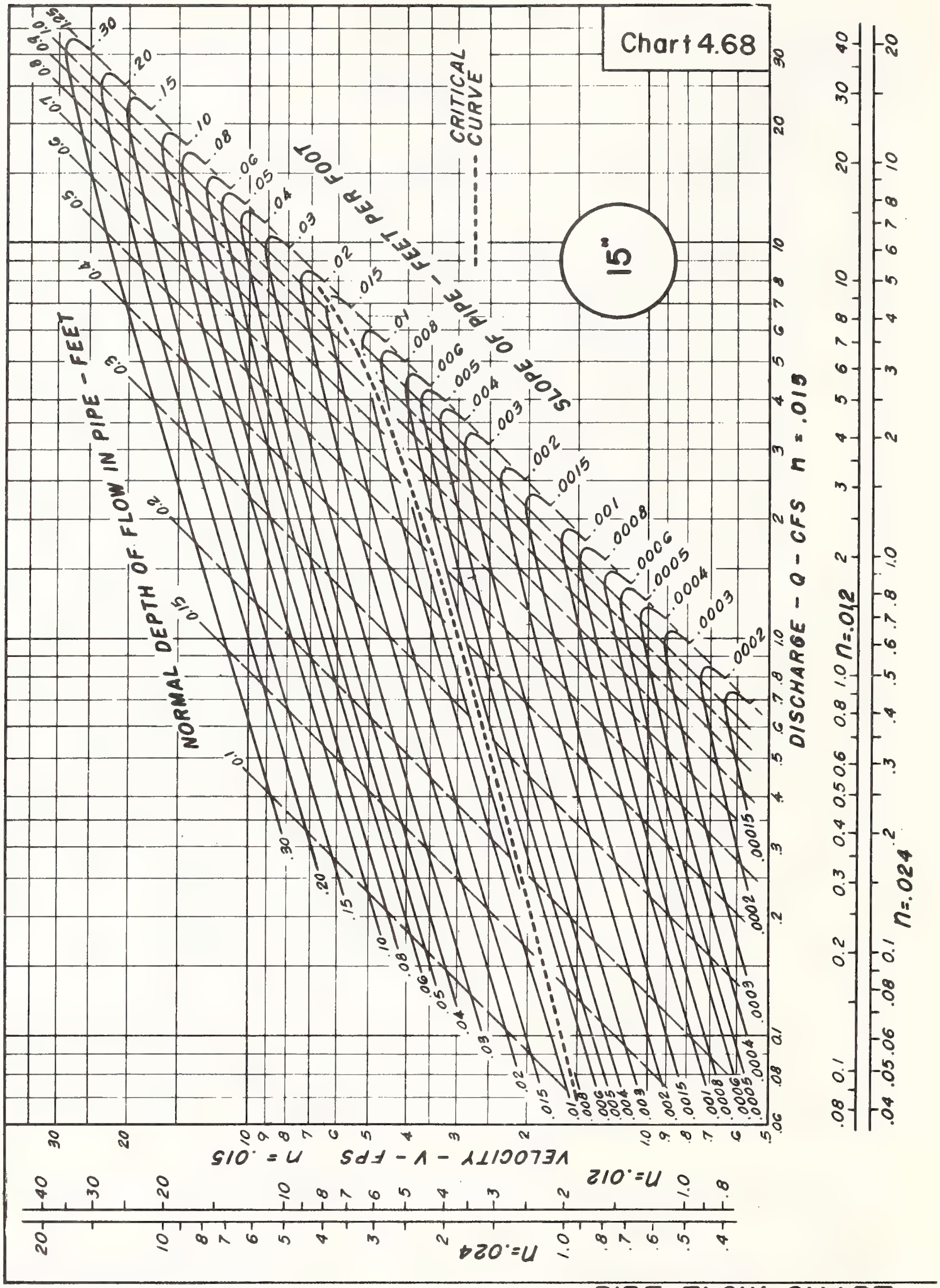
The second set of charts for circular-pipe channels, Nos. 4.84 - 4.92, differ from Charts 4.67 - 4.83 in that they require the use of several charts for solving the Manning equation. The charts contain curves for standard sizes of pipe up to 15 feet in diameter, for values of n = 0.011, 0.012, and 0.025. The relations of friction slope, discharge, velocity, and pipe diameter for pipes with n = 0.025 are given on Chart 4.84; similar relations for pipes with n =0.011 and 0.012 are given on Charts 4.85 and 4.86. Ratios for computation of part-full pipe flow are given on Chart 4.87. Chart 4.88 shows critical depth and Chart 4.89 shows specific head at critical depth; both are independent of the n value of the pipe. Chart 4.90 shows the critical slope for pipes with n = 0.025, and Charts 4.91 and 4.92 show critical slope for pipes with n values of 0.011

and 0.012.

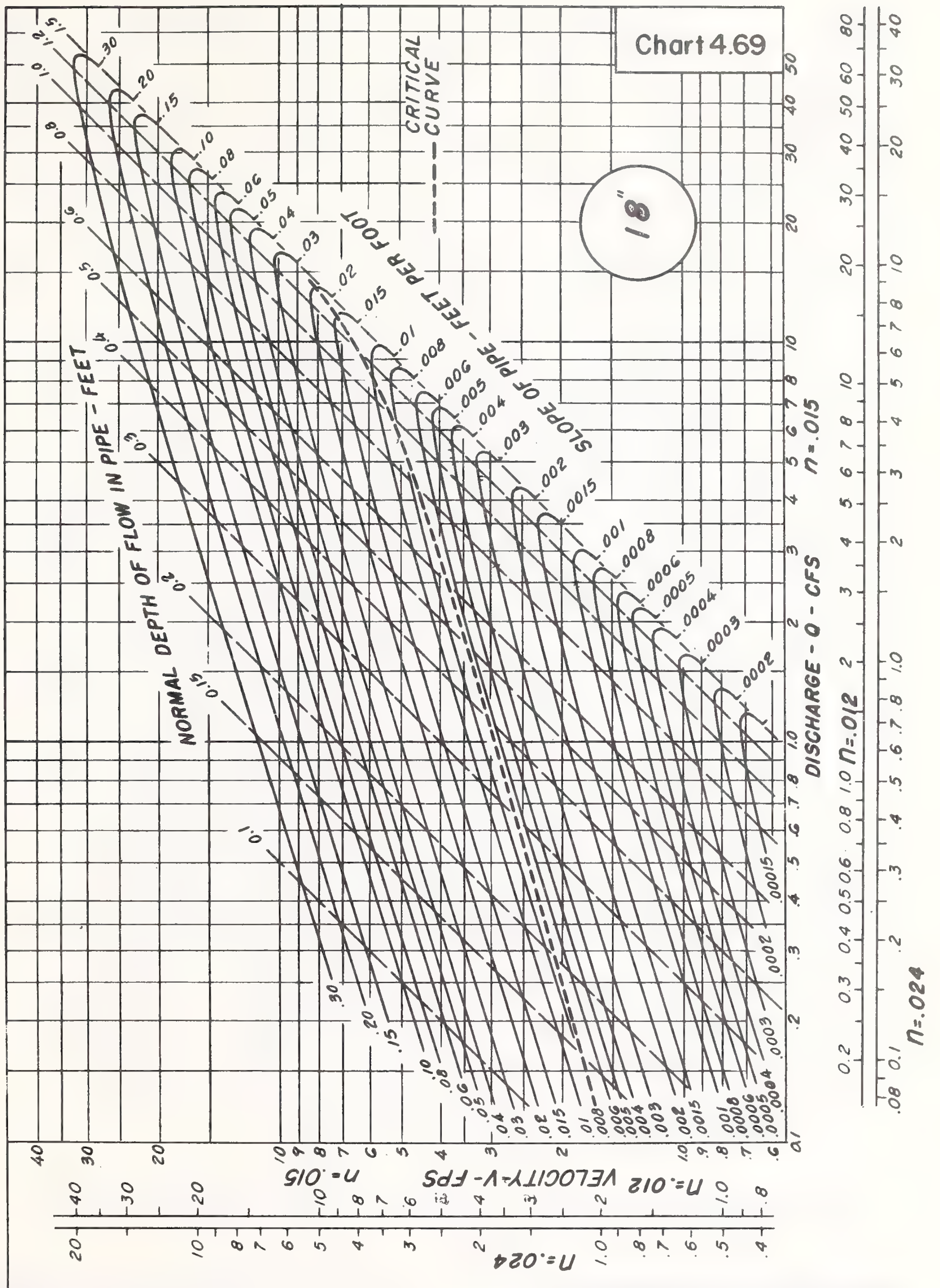
Table 4.34 describes the geometric elements for circular sections which are flowing partially full.



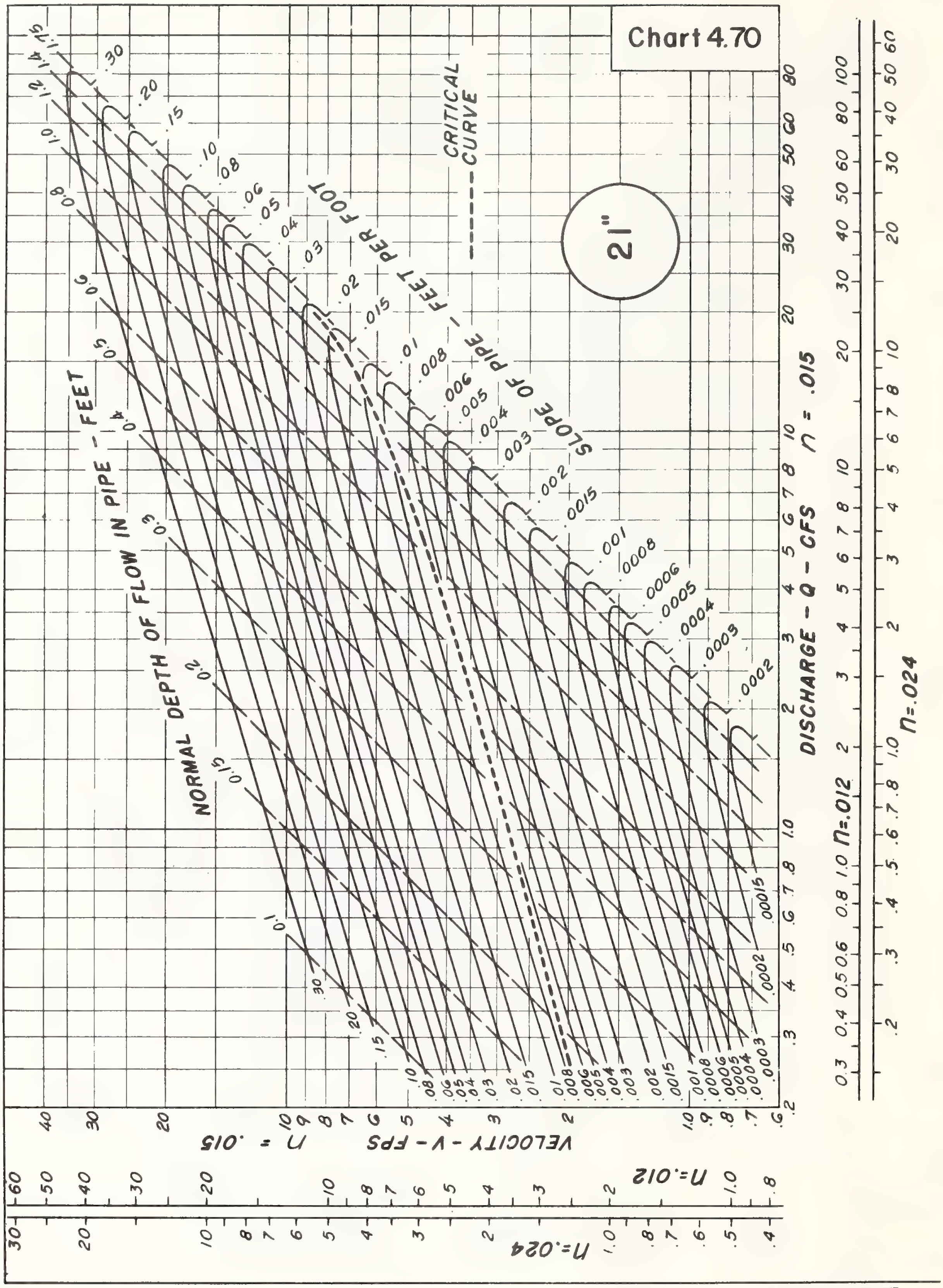
PIPE FLOW CHART
12-INCH DIAMETER



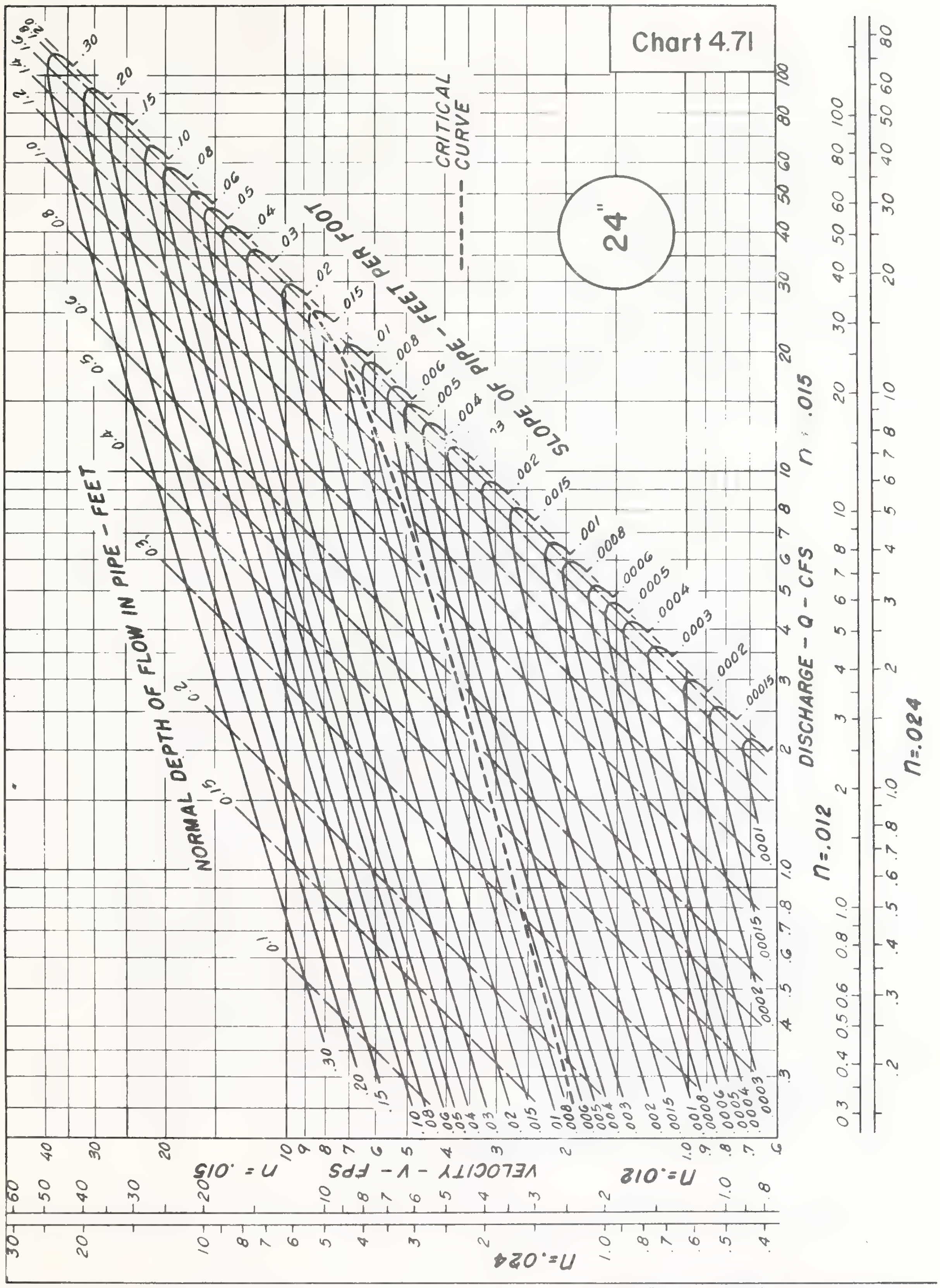
PIPE FLOW CHART
15-INCH DIAMETER



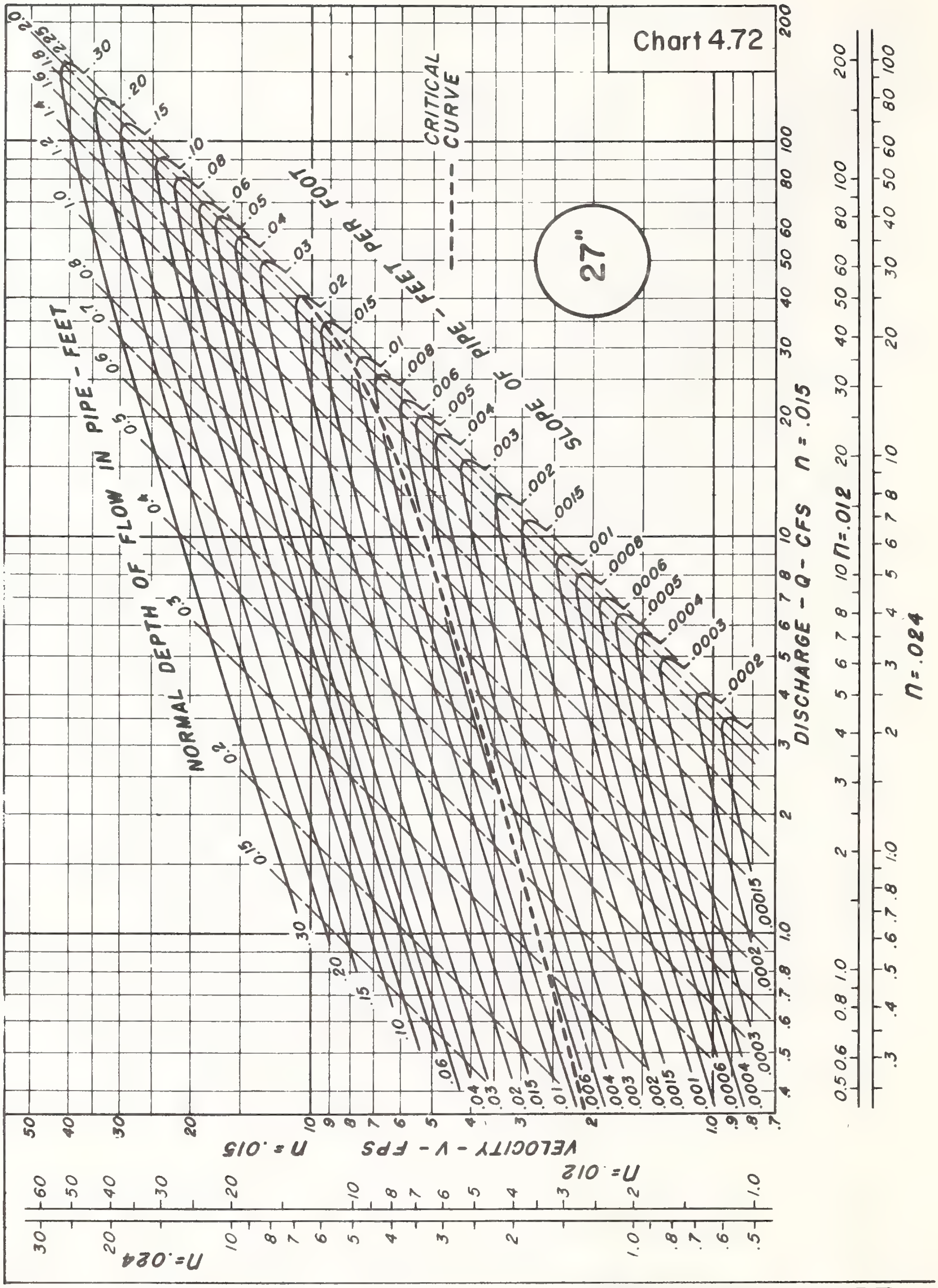
PIPE FLOW CHART
18-INCH DIAMETER



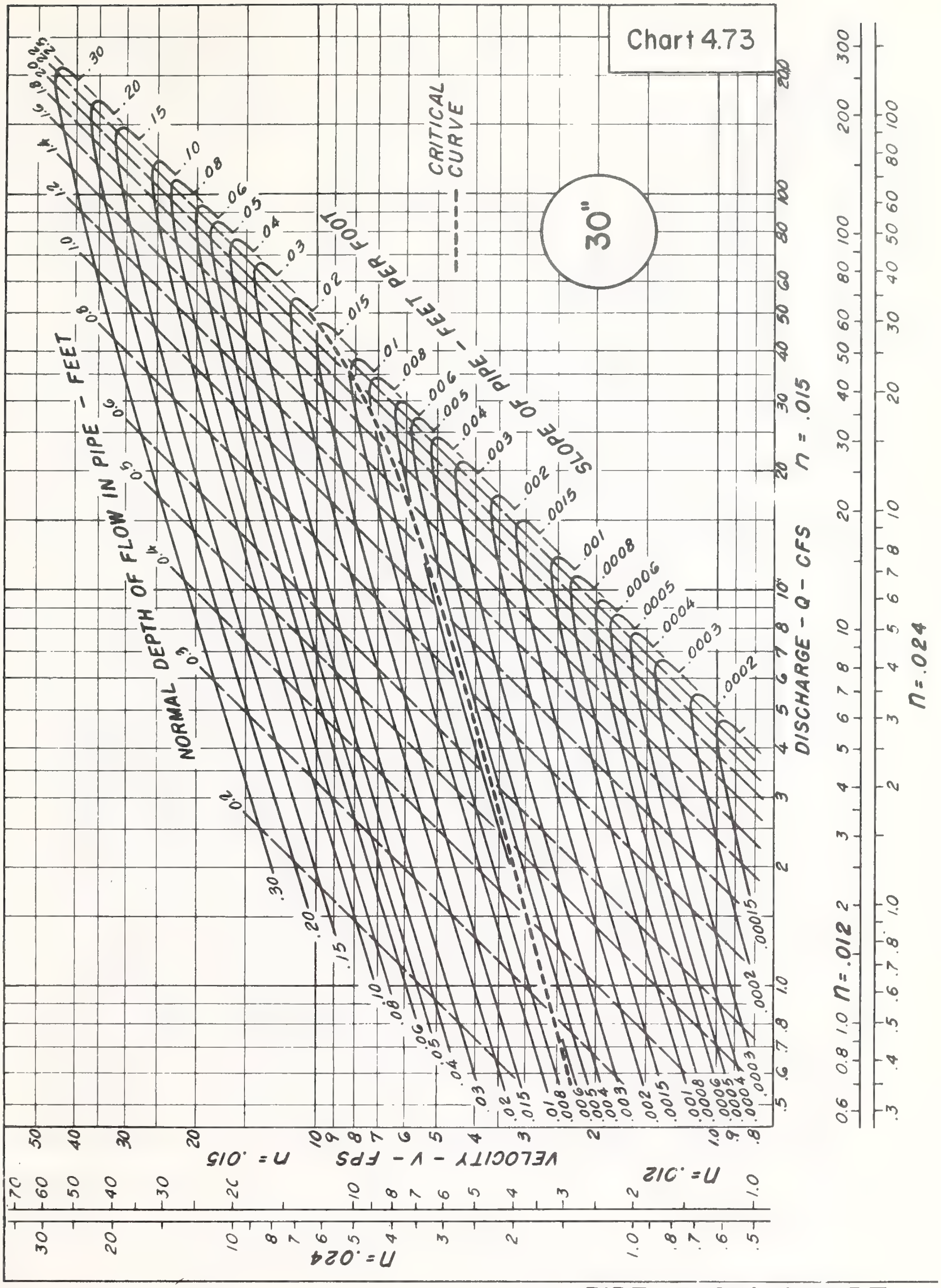
PIPE FLOW CHART
21-INCH DIAMETER



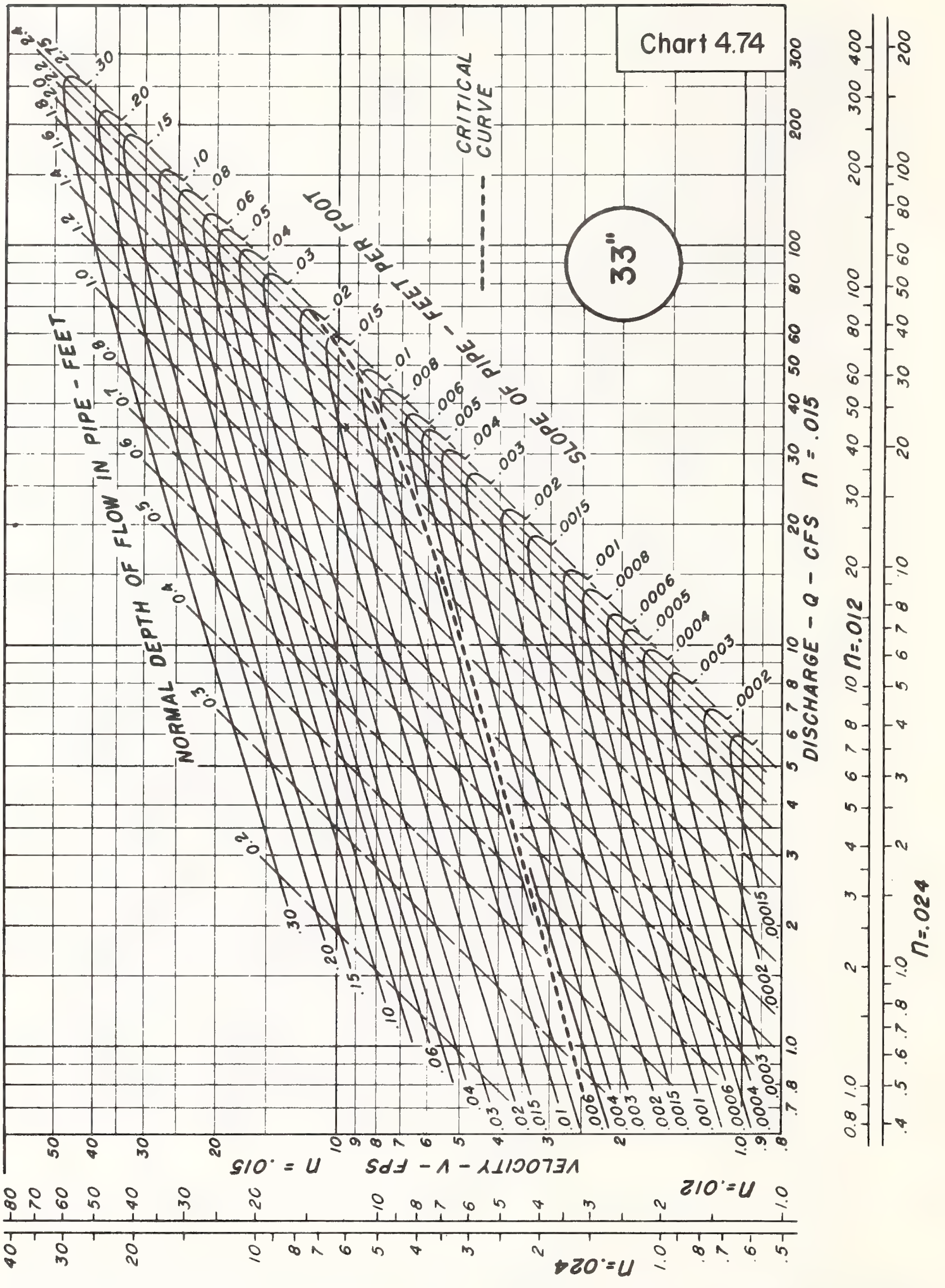
PIPE FLOW CHART
24-INCH DIAMETER



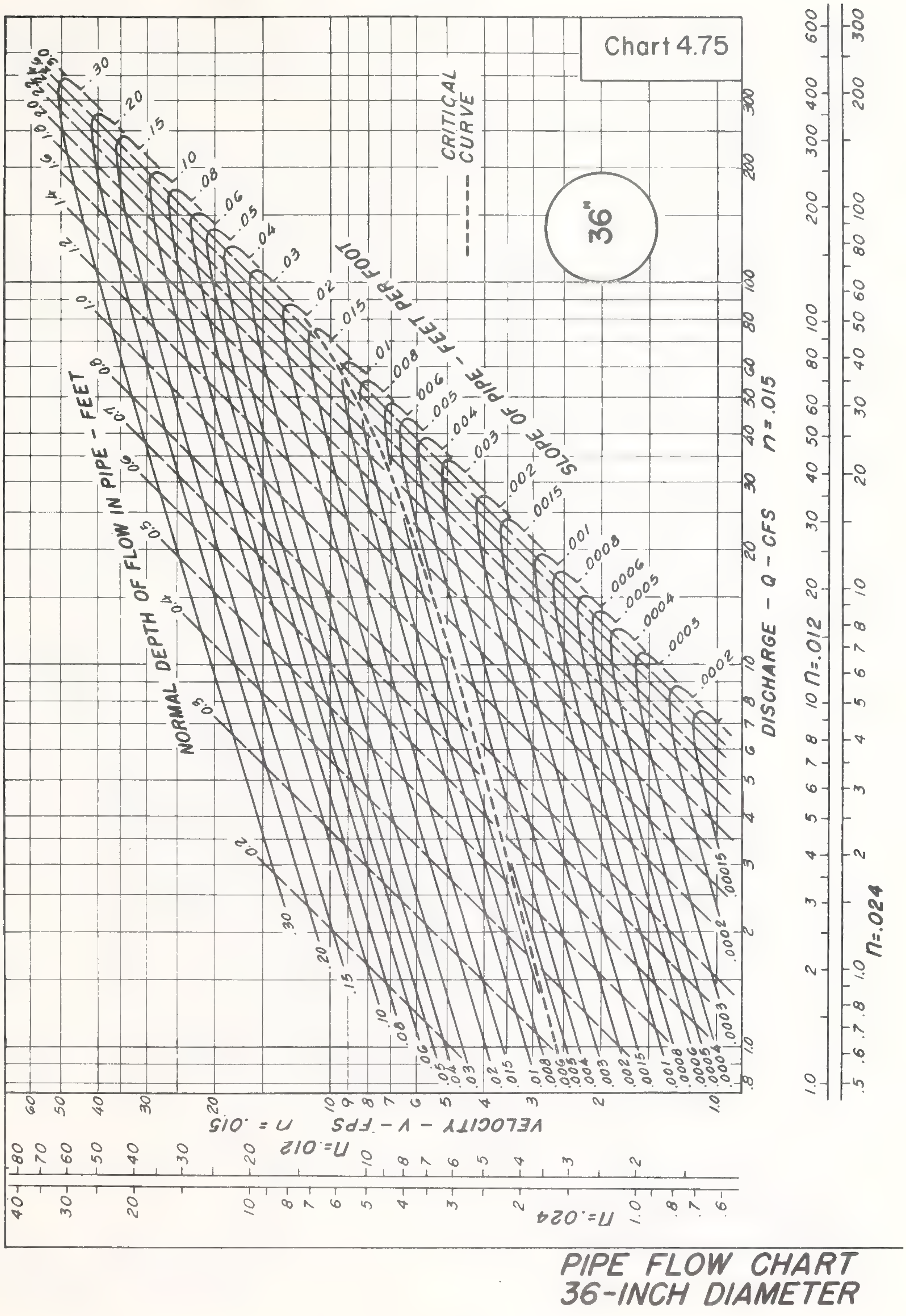
PIPE FLOW CHART
27-INCH DIAMETER



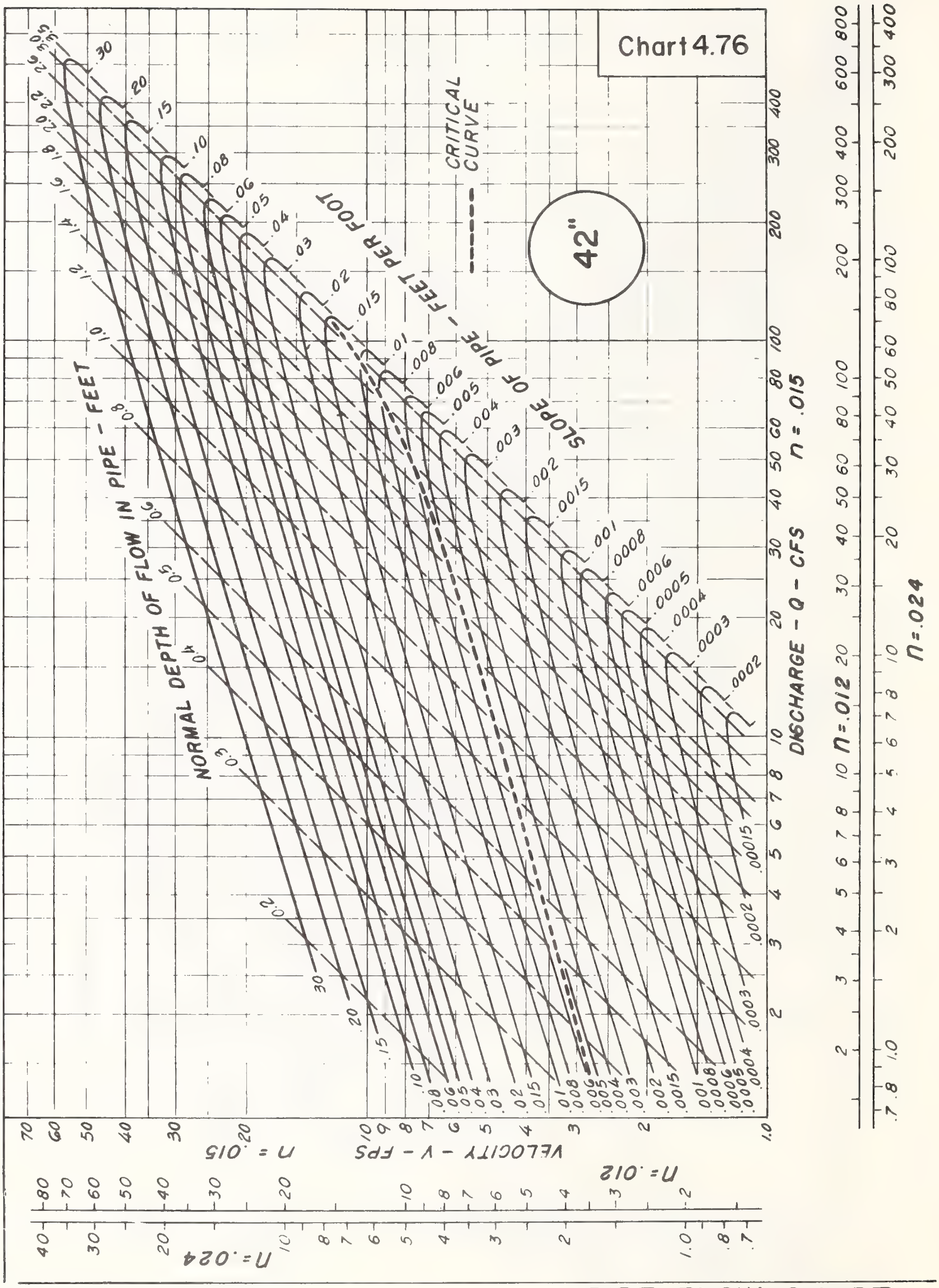
PIPE FLOW CHART
30-INCH DIAMETER



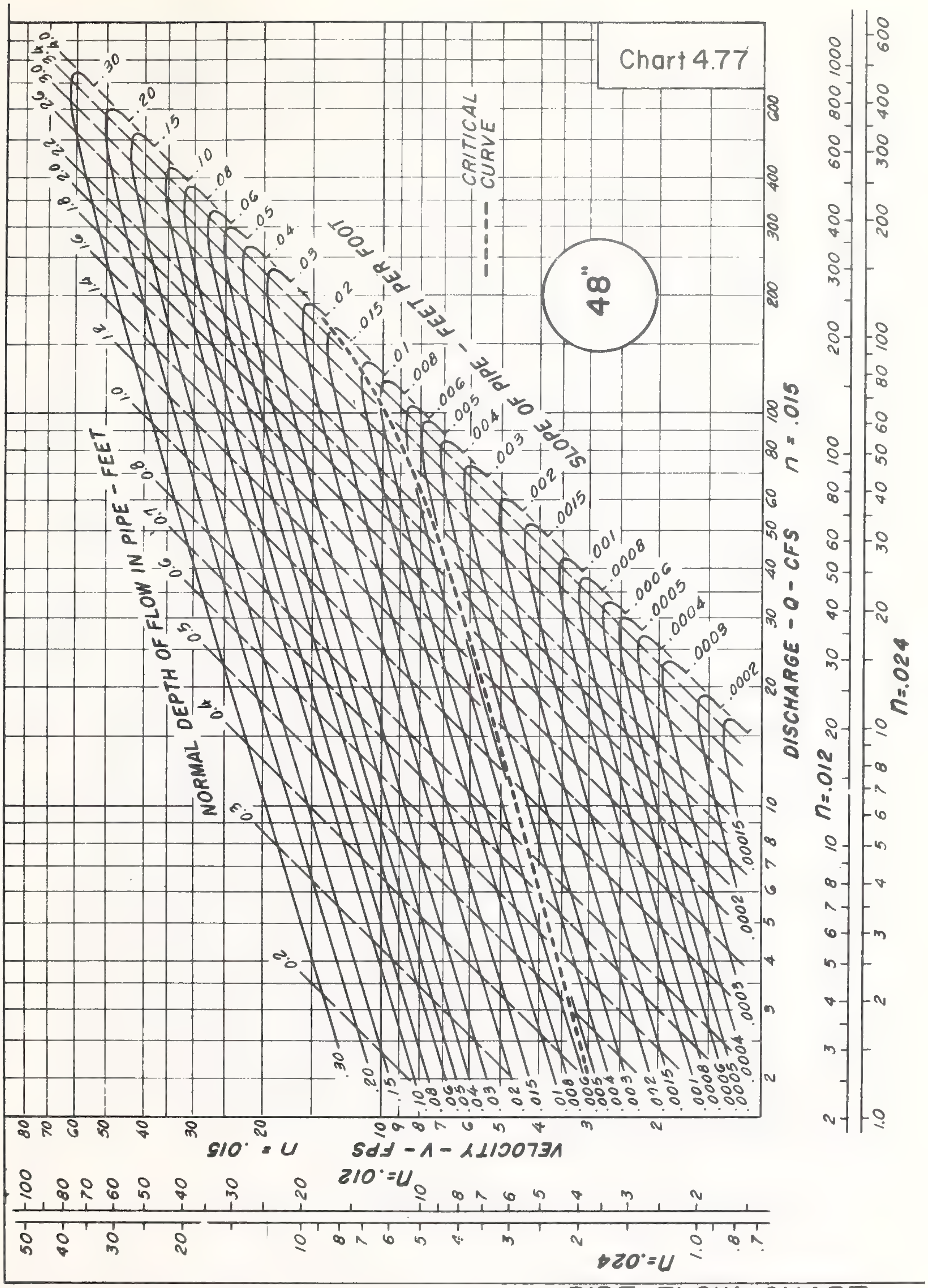
PIPE FLOW CHART
33-INCH DIAMETER



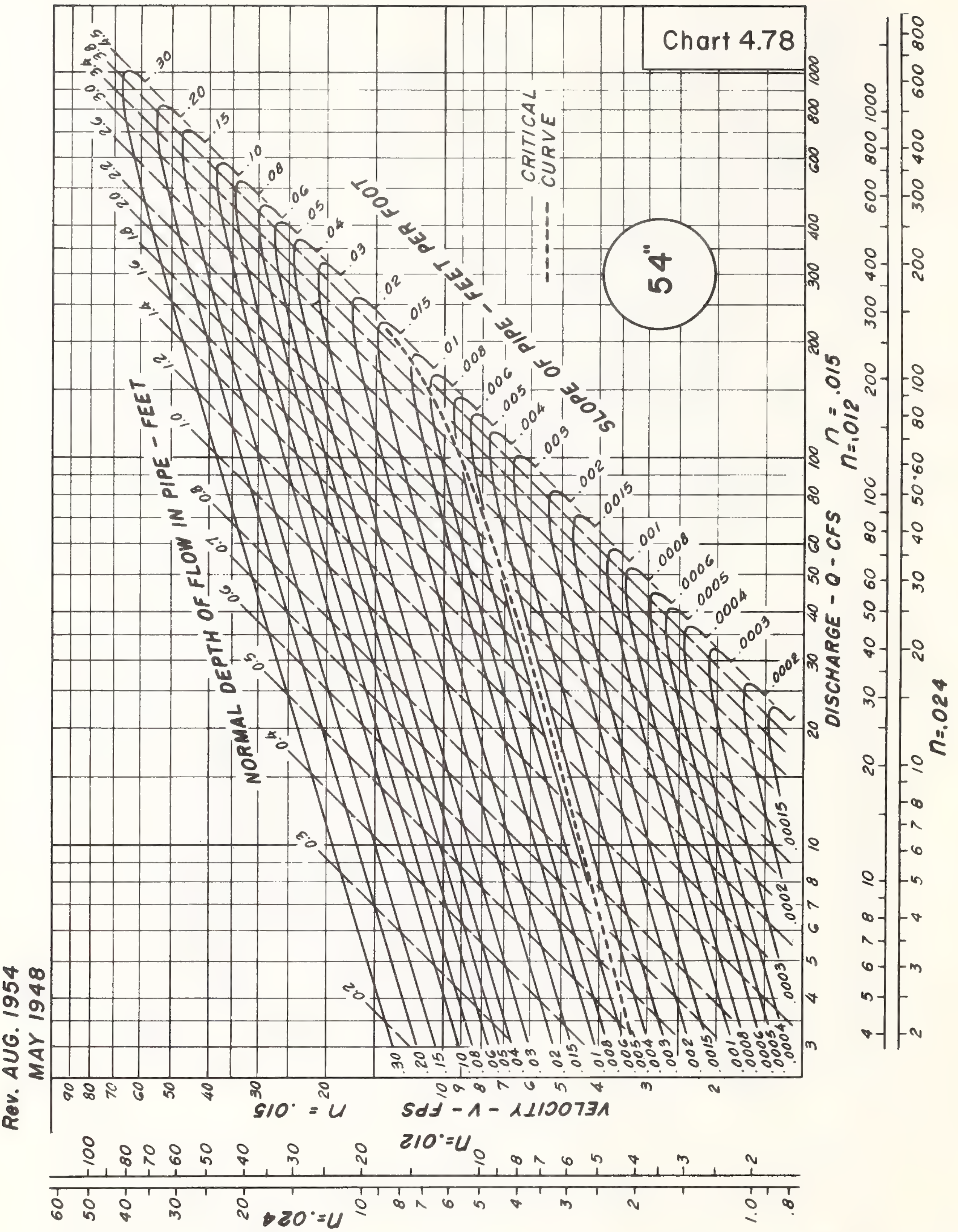
PIPE FLOW CHART
36-INCH DIAMETER



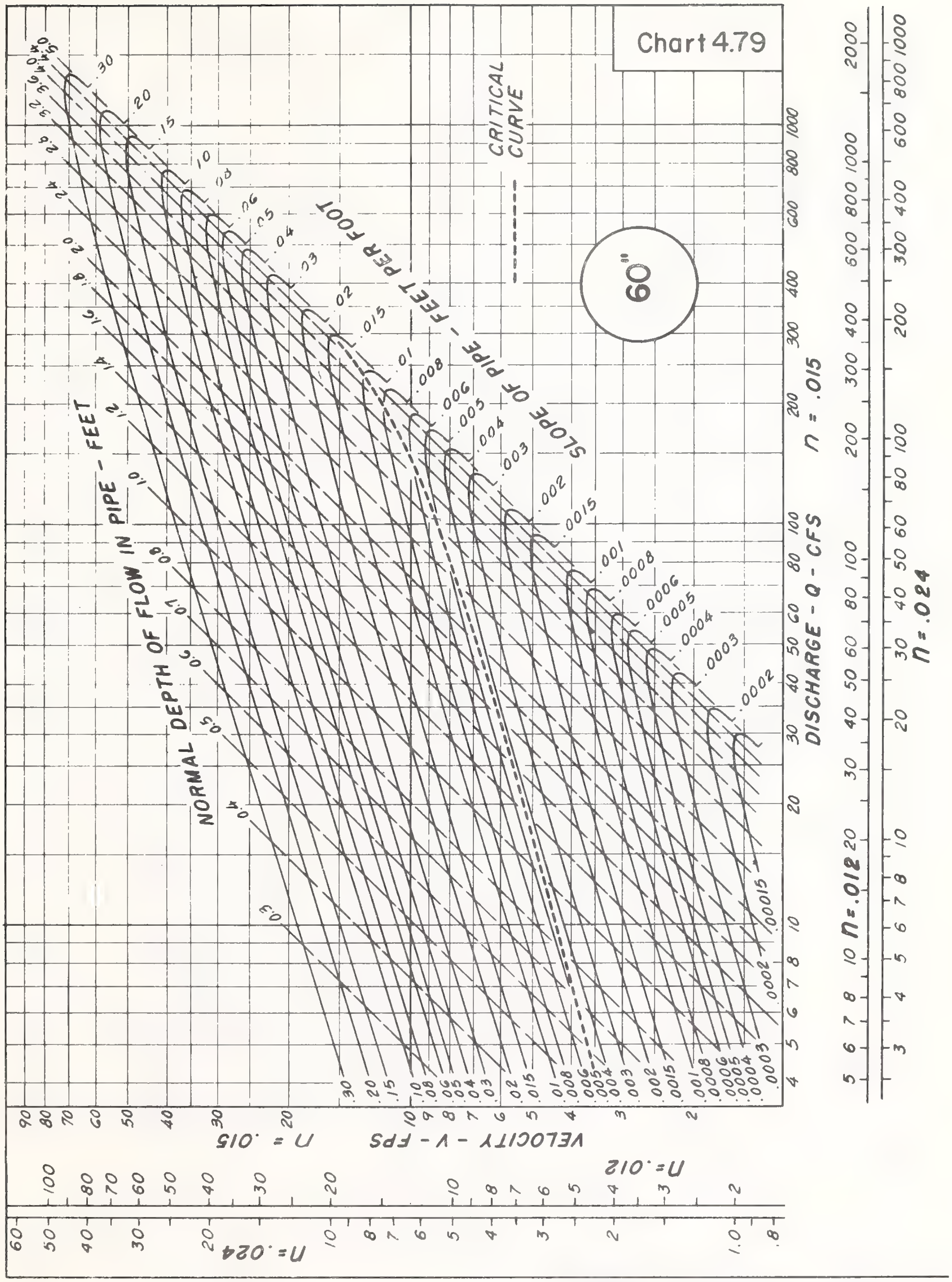
PIPE FLOW CHART
42-INCH DIAMETER



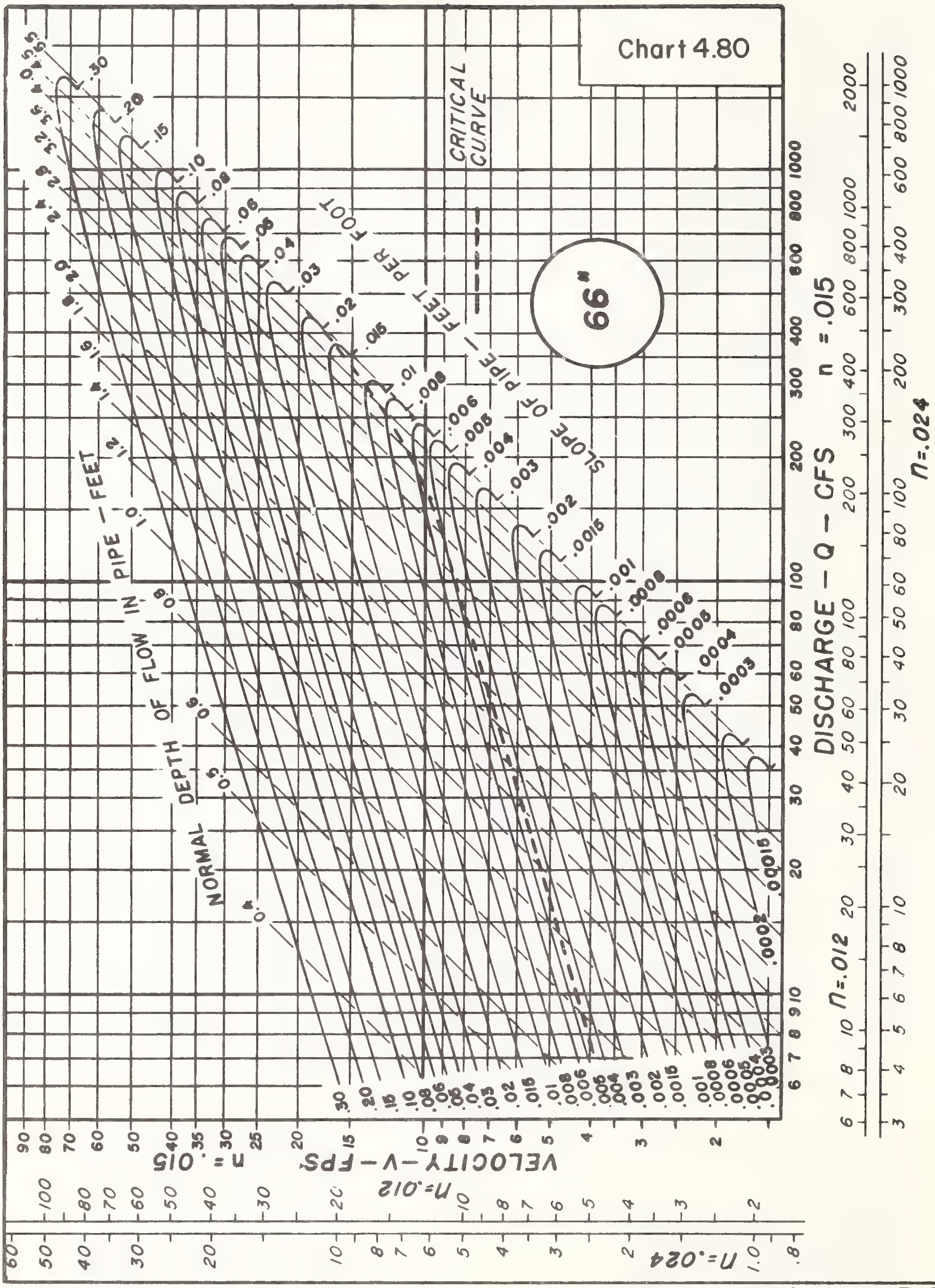
PIPE FLOW CHART
48-INCH DIAMETER



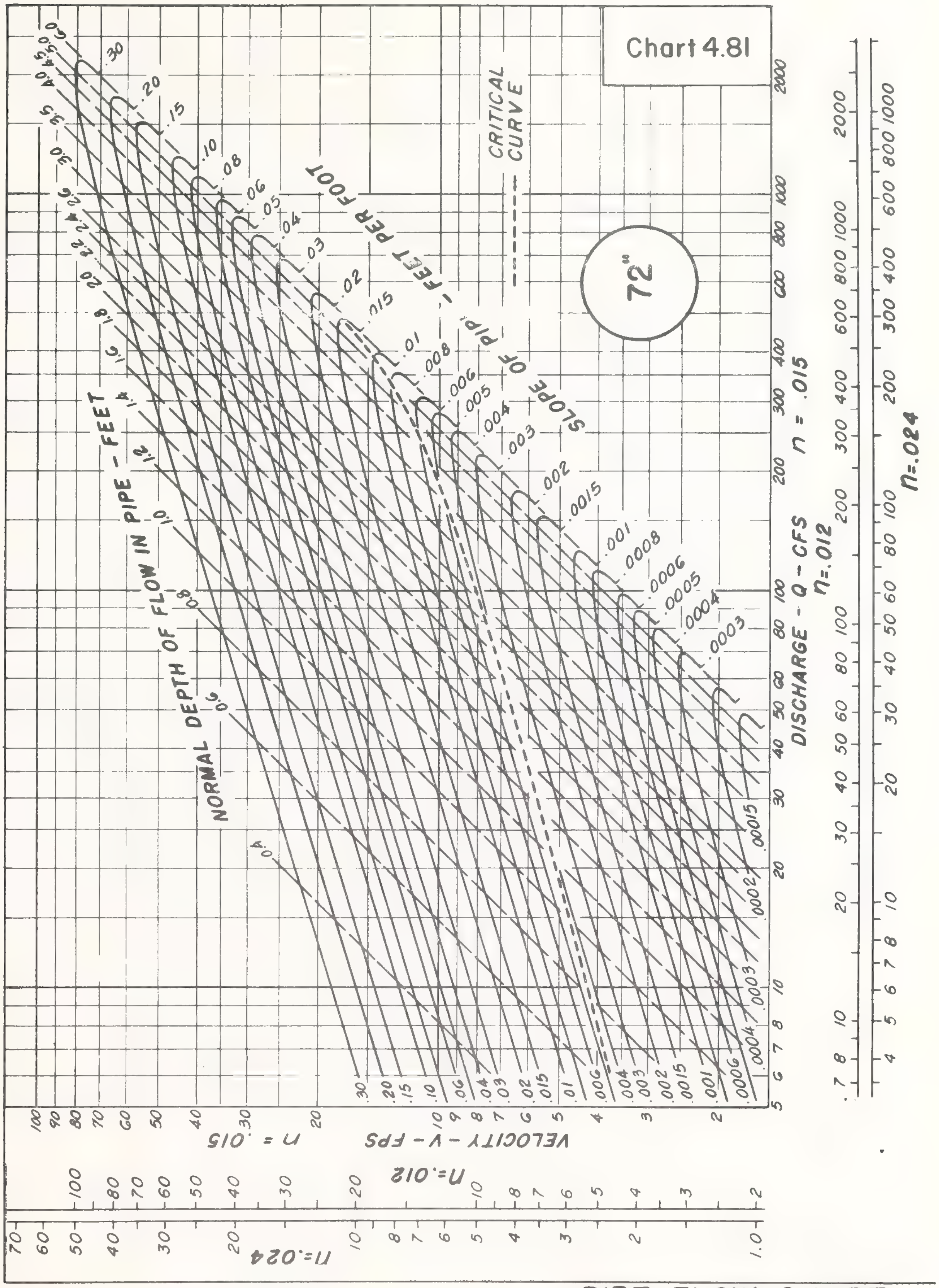
PIPE FLOW CHART
54-INCH DIAMETER



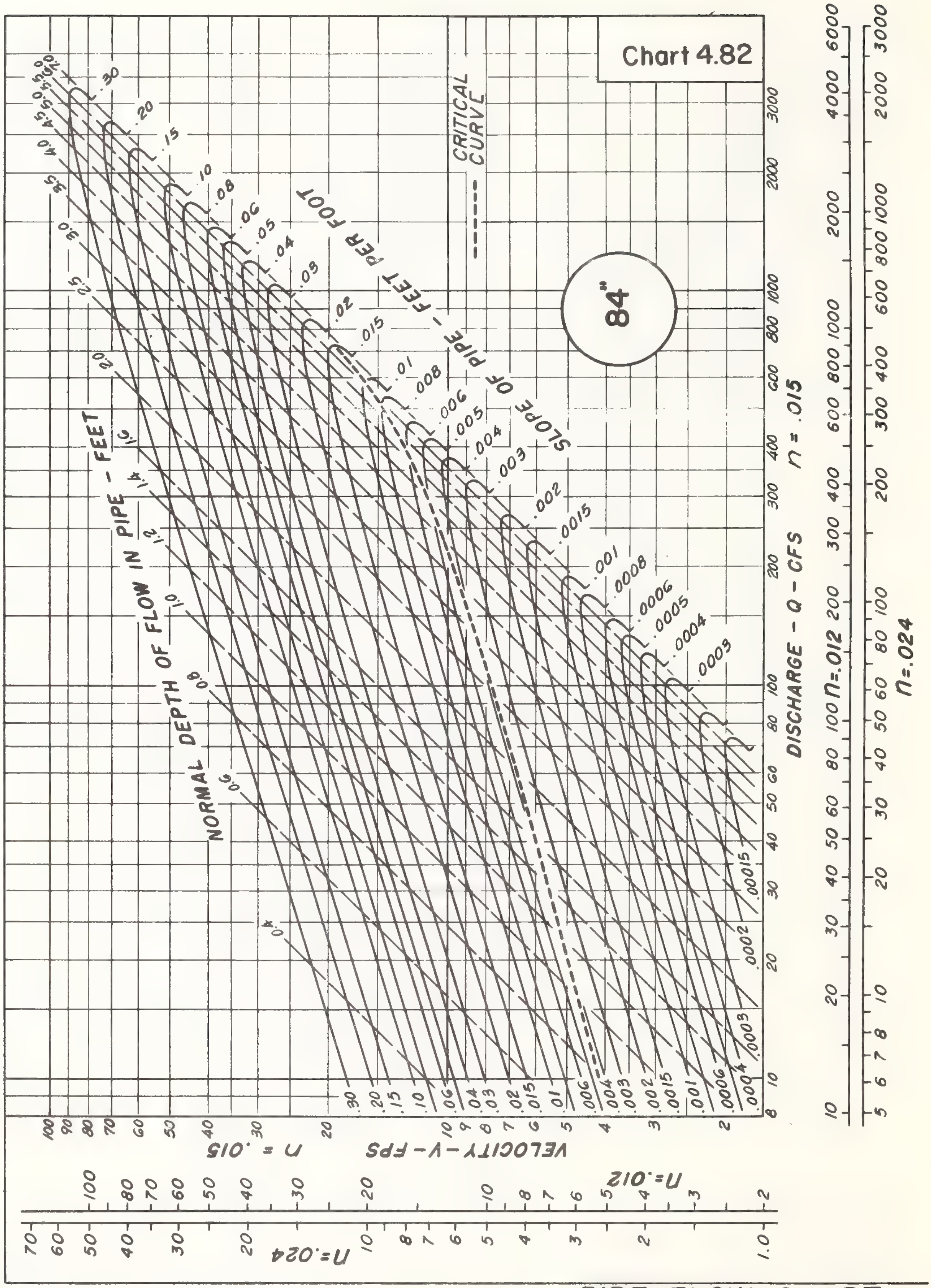
PIPE FLOW CHART
60-INCH DIAMETER



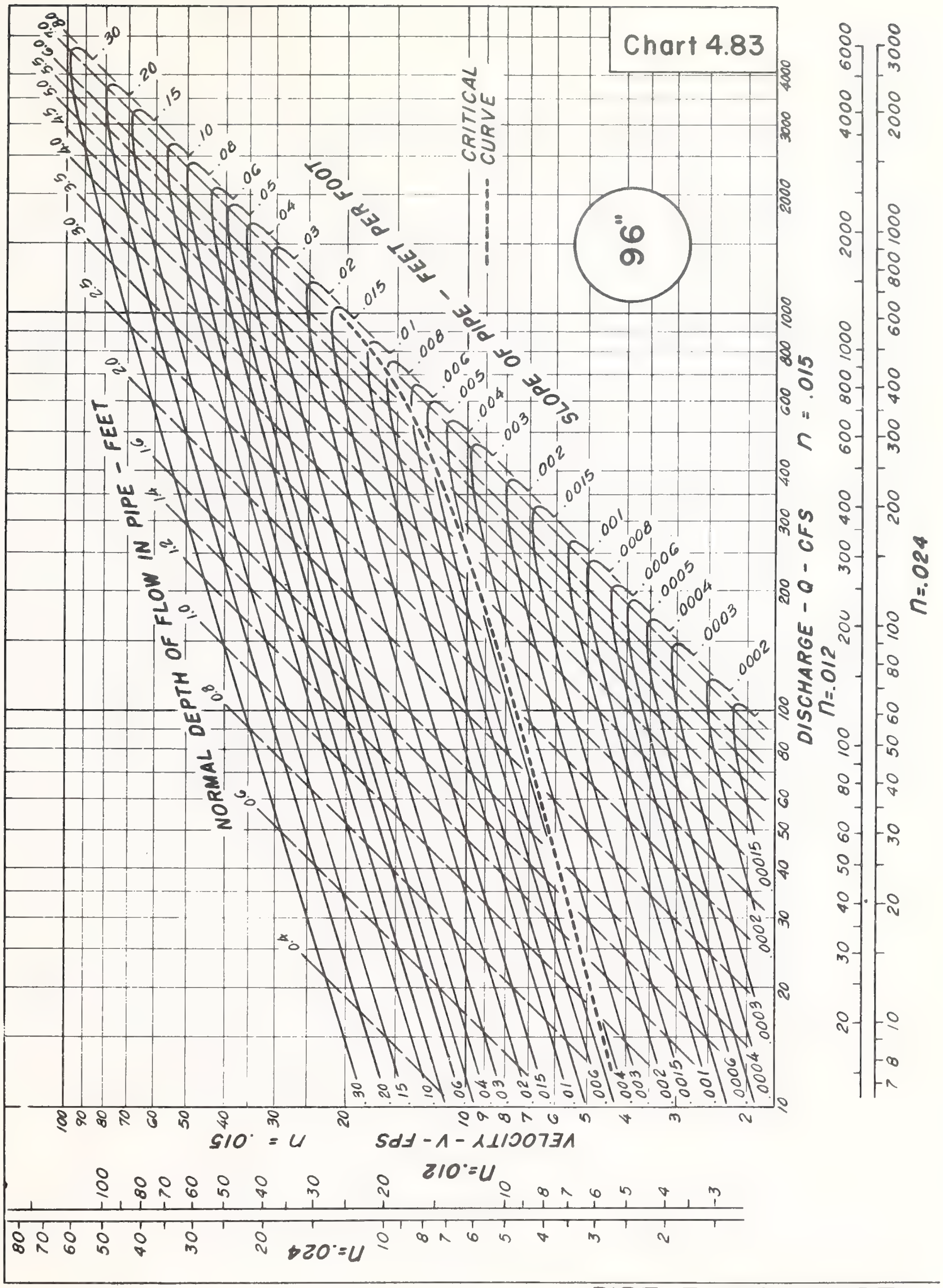
PIPE FLOW CHART
66-INCH DIAMETER



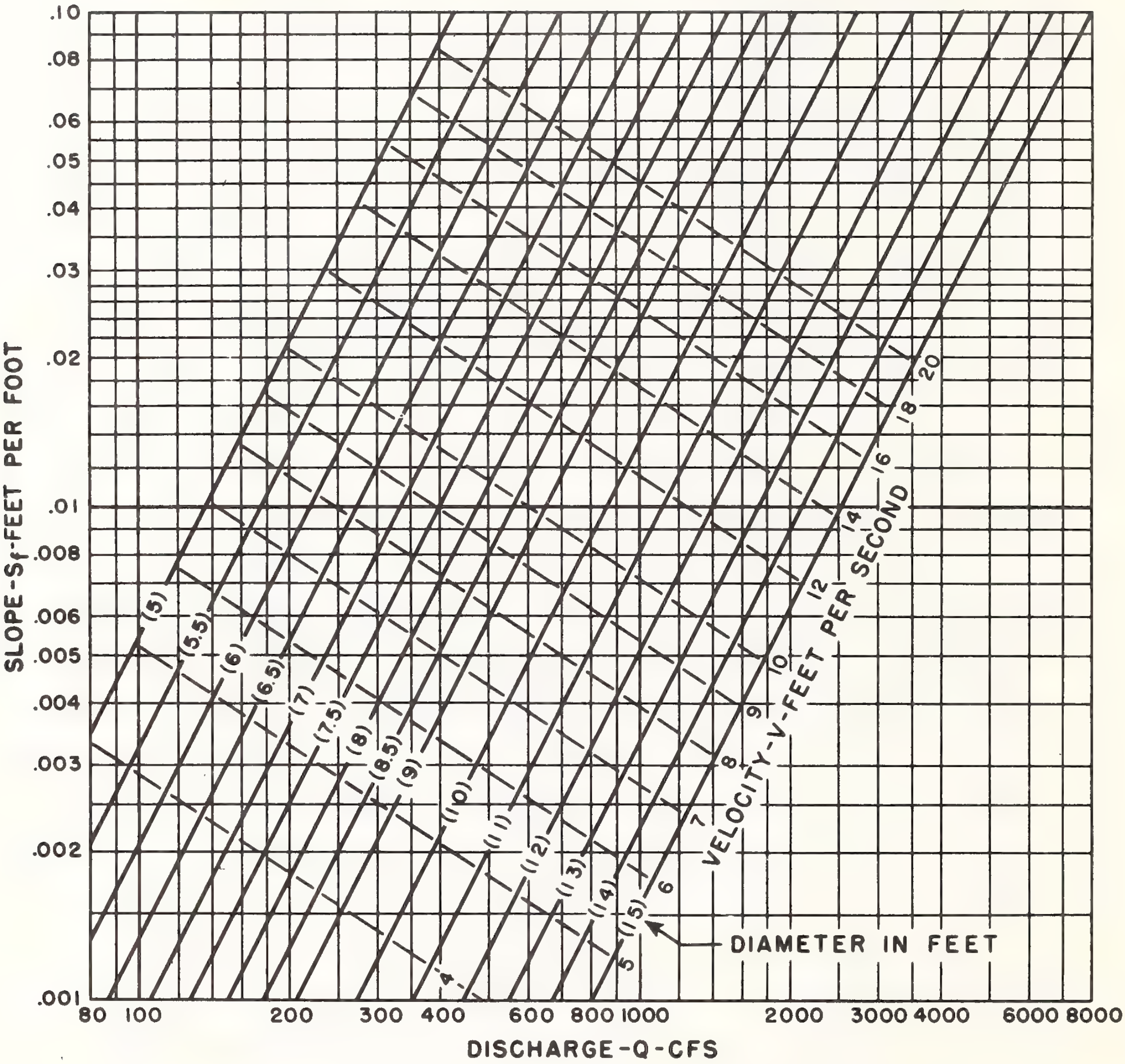
PIPE FLOW CHART
72-INCH DIAMETER



PIPE FLOW CHART
84-INCH DIAMETER

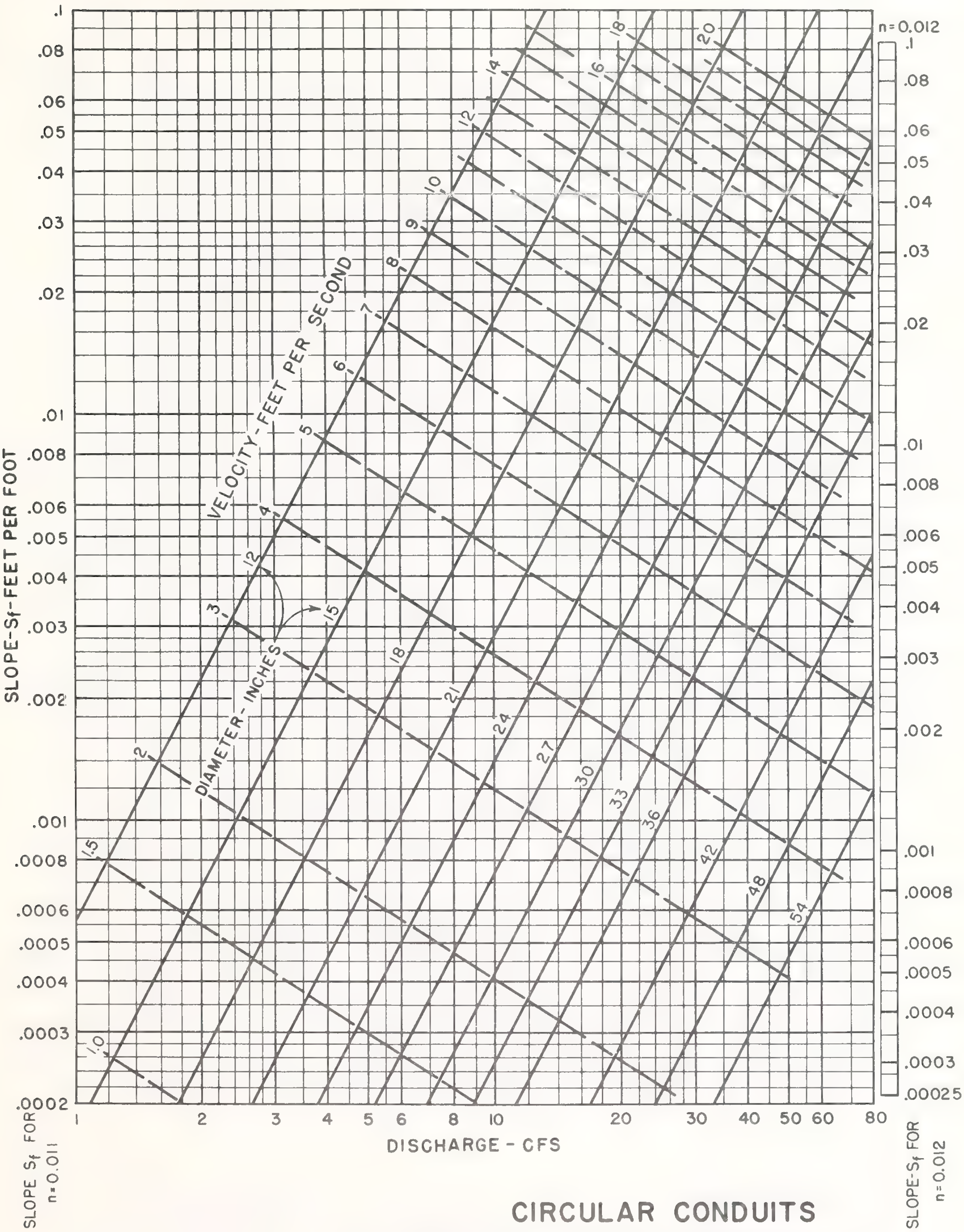


PIPE FLOW CHART
96-INCH DIAMETER

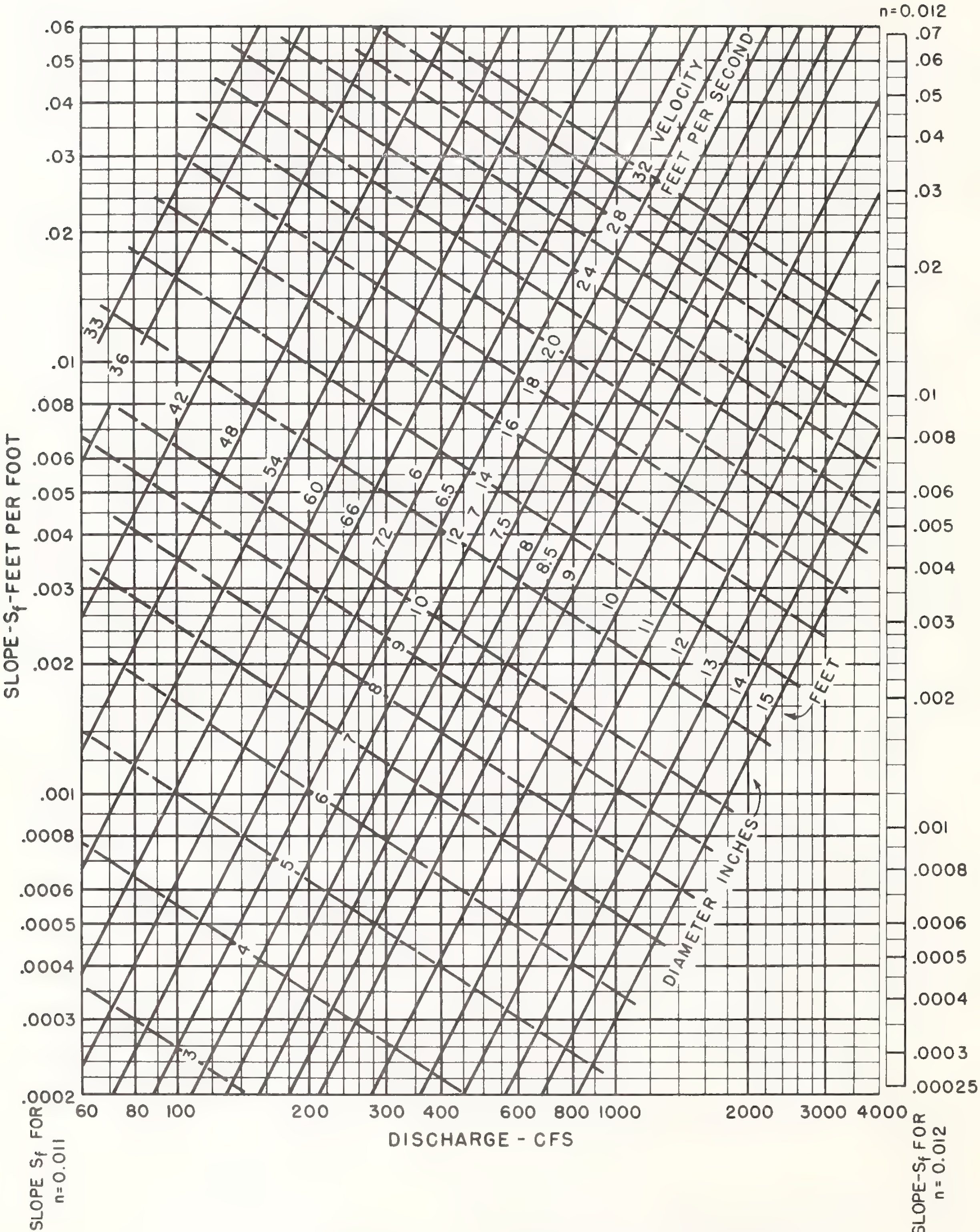


CIRCULAR C. M. PIPE
FRICTION SLOPE FLOWING FULL
n=0.025

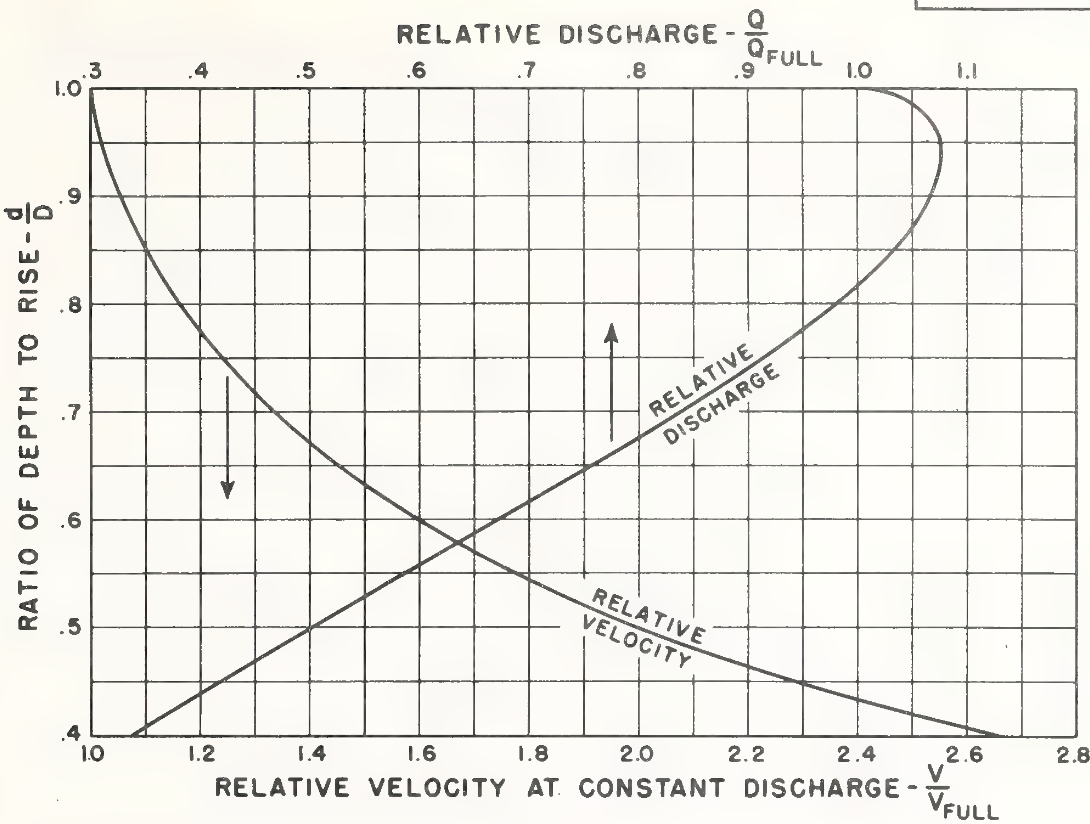
Chart 4.85

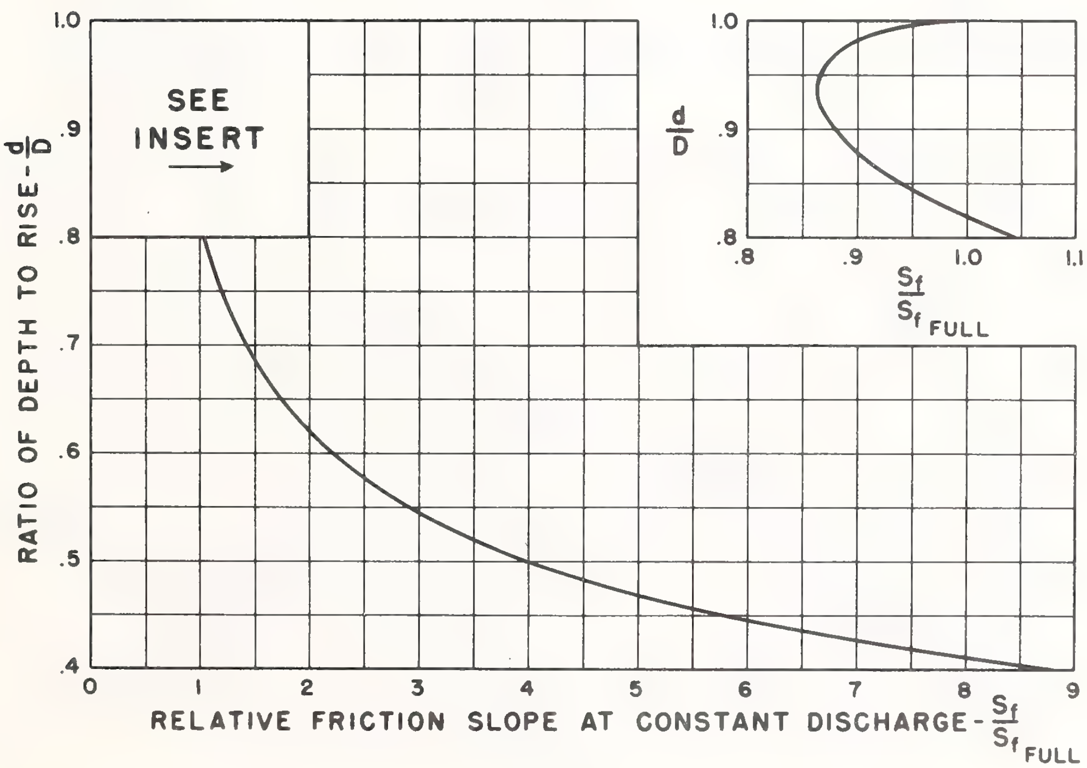


CIRCULAR CONDUITS
FRICTION SLOPE FLOWING FULL
n=0.011(and 0.012)

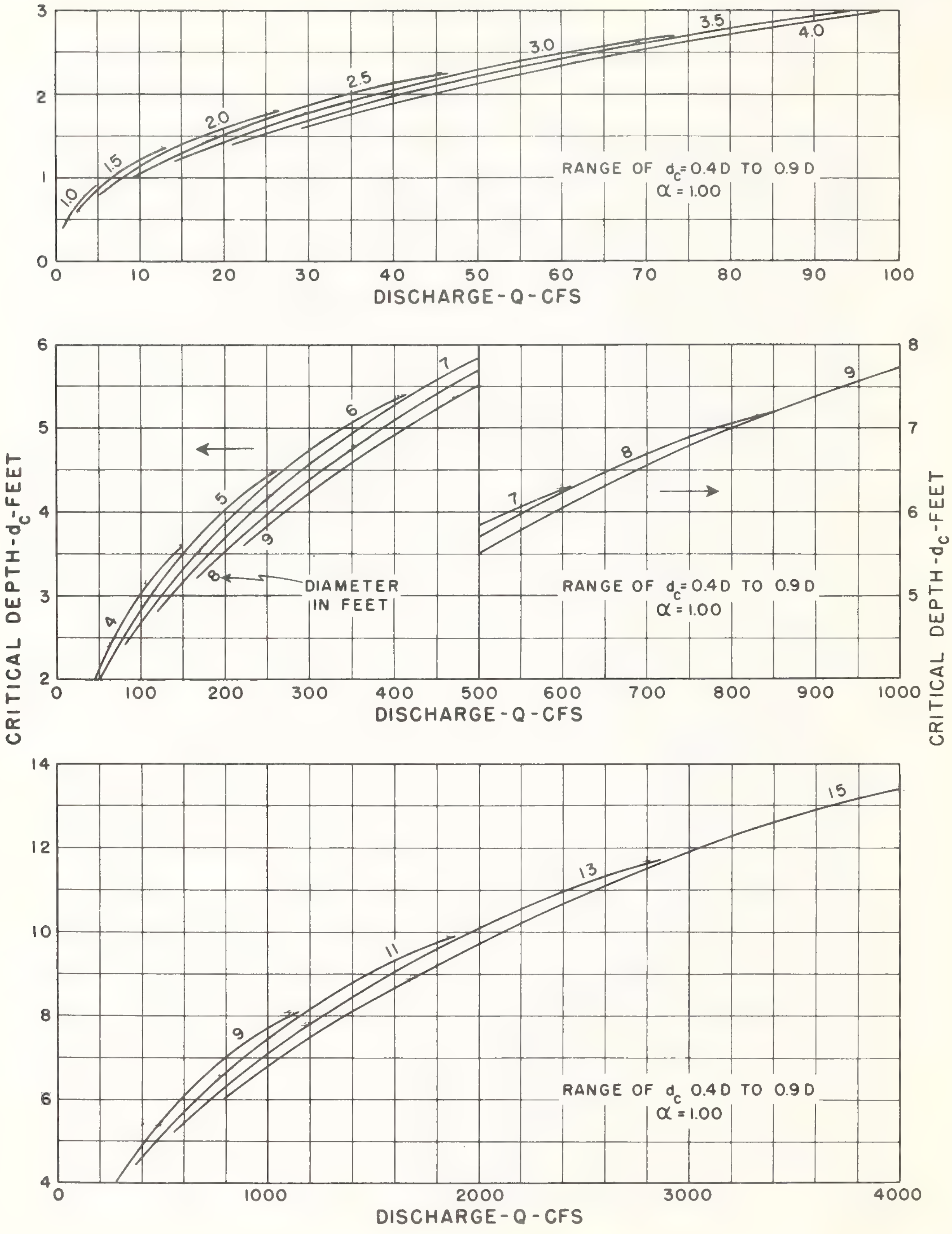


CIRCULAR CONDUITS
FRICTION SLOPE FLOWING FULL
n=0.011 (and 0.012)



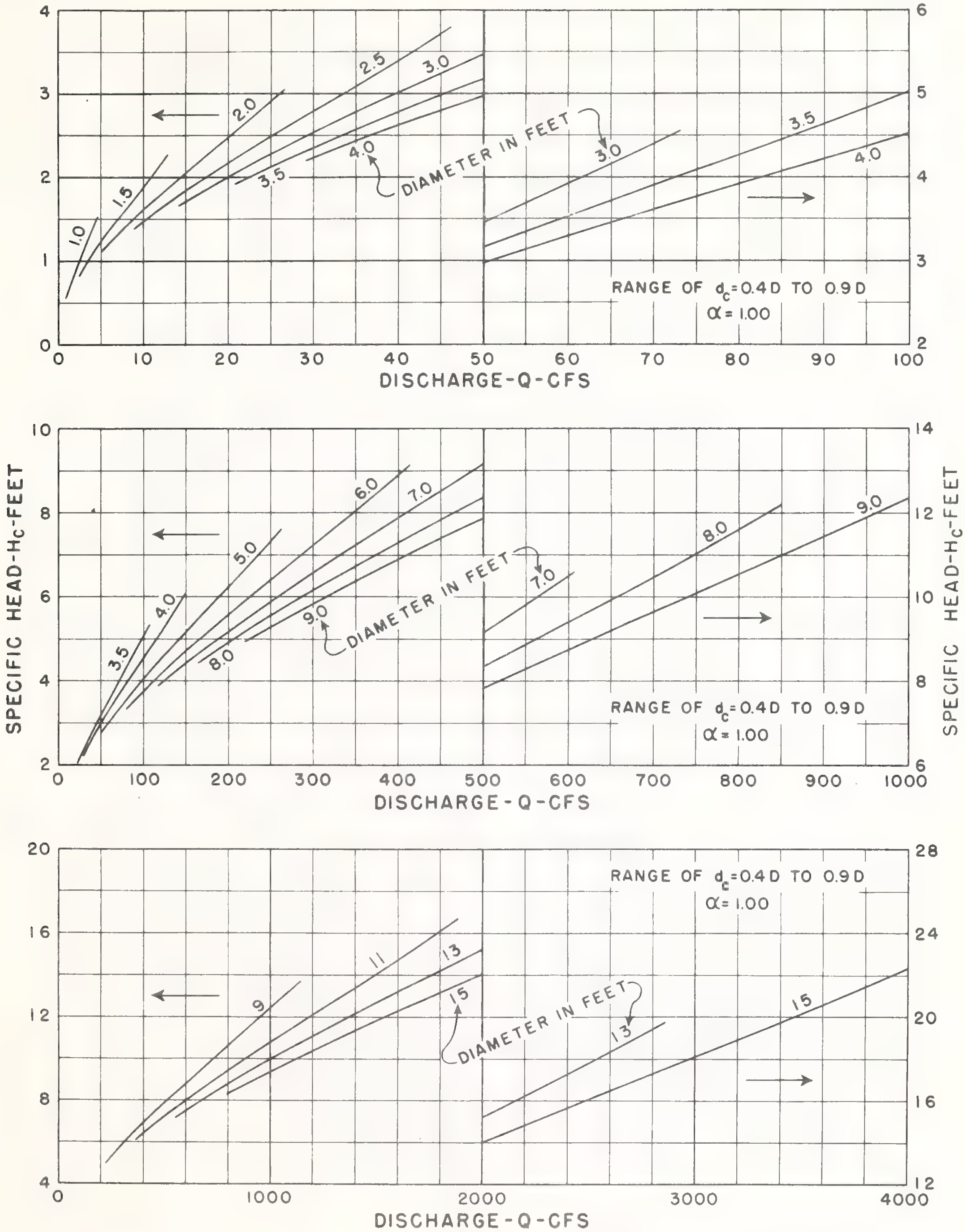


CIRCULAR PIPE
PART FULL FLOW



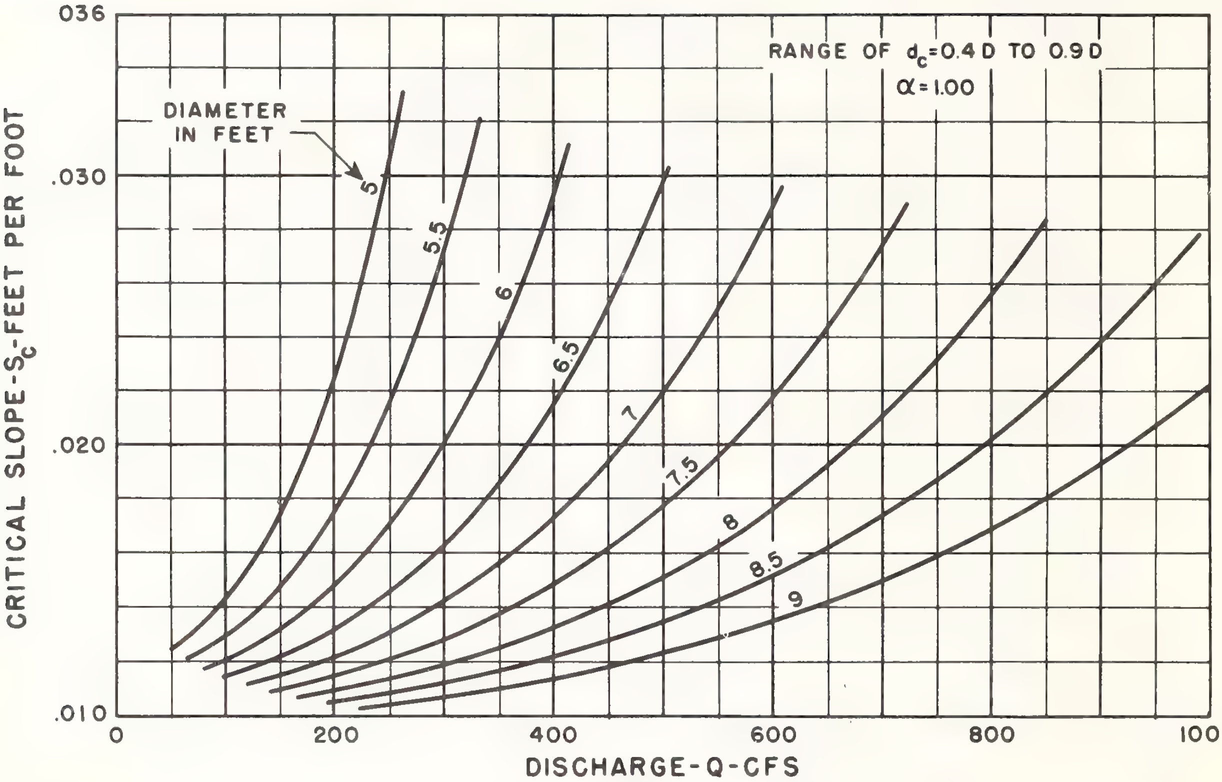
CIRCULAR PIPE
CRITICAL DEPTH

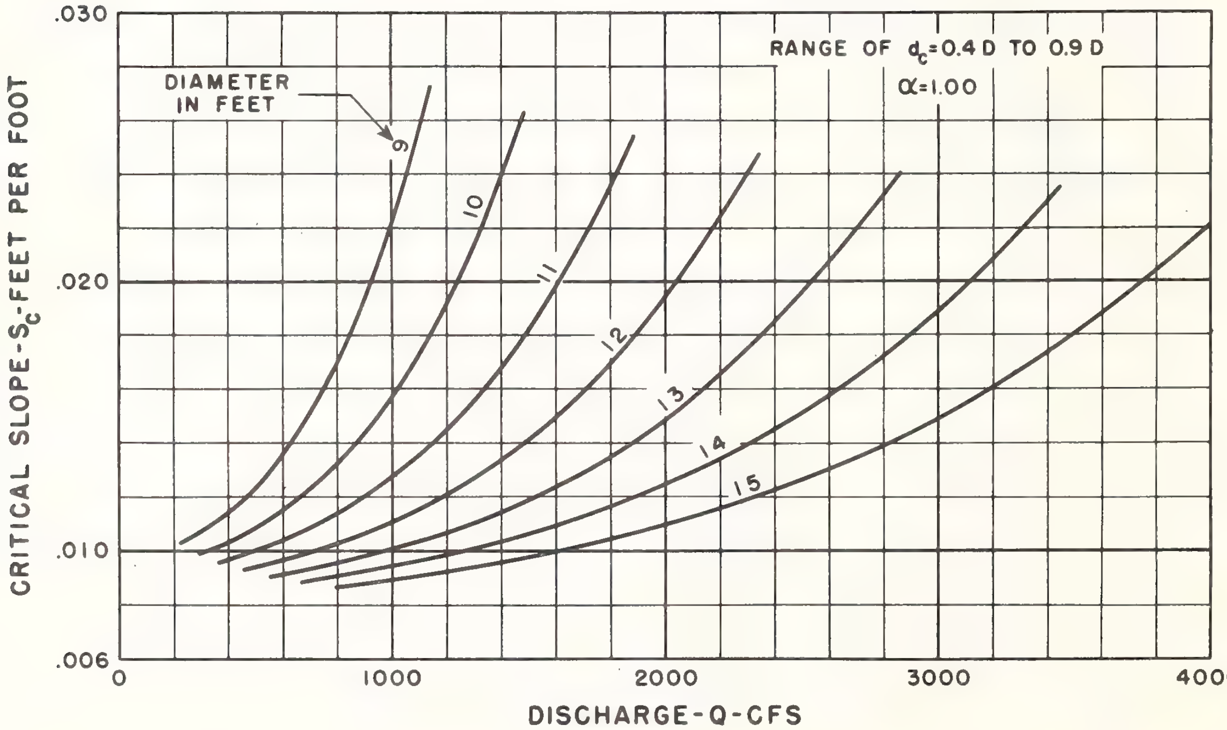
Chart 4.89



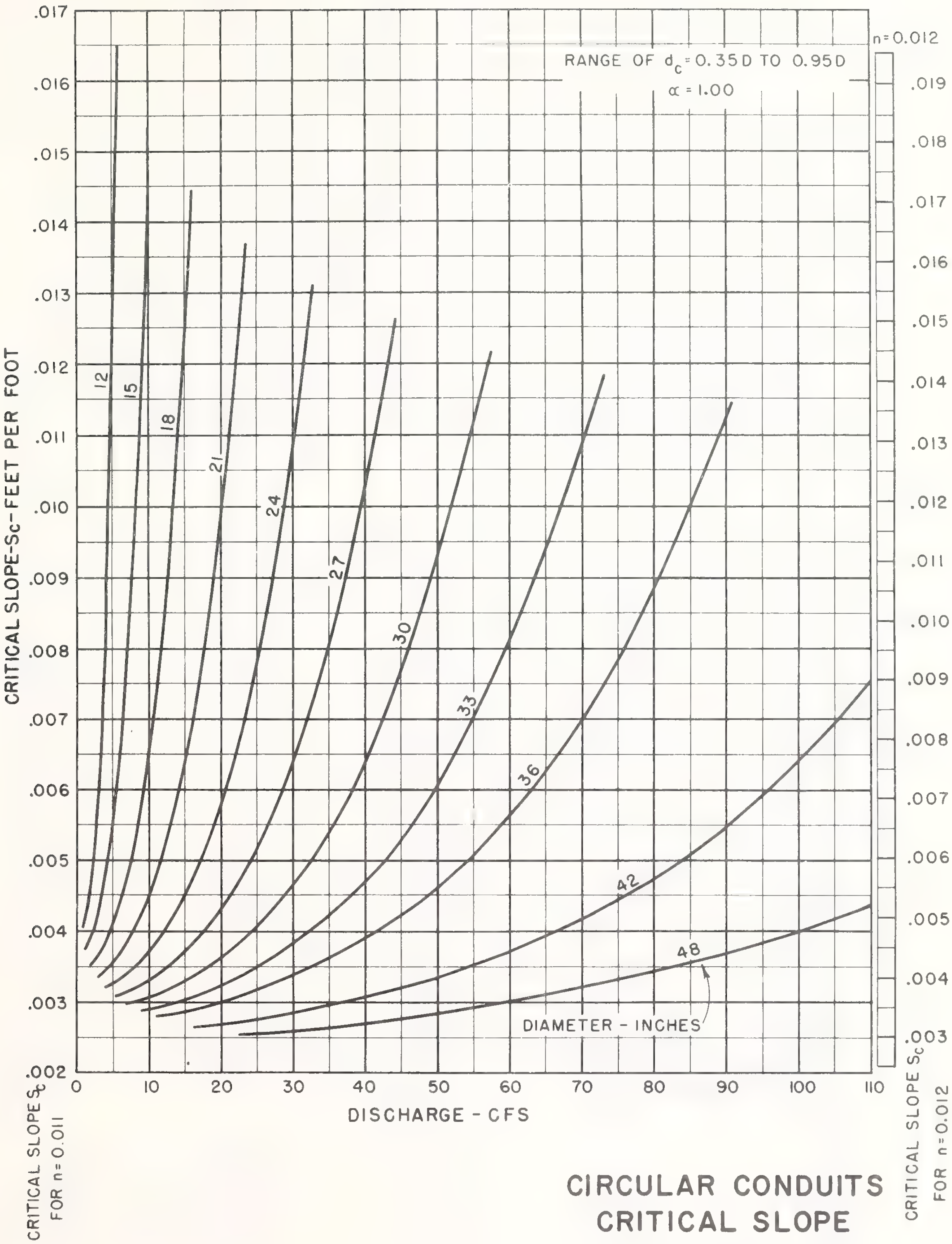
CIRCULAR PIPE
SPECIFIC HEAD AT CRITICAL DEPTH

Chart 4.90

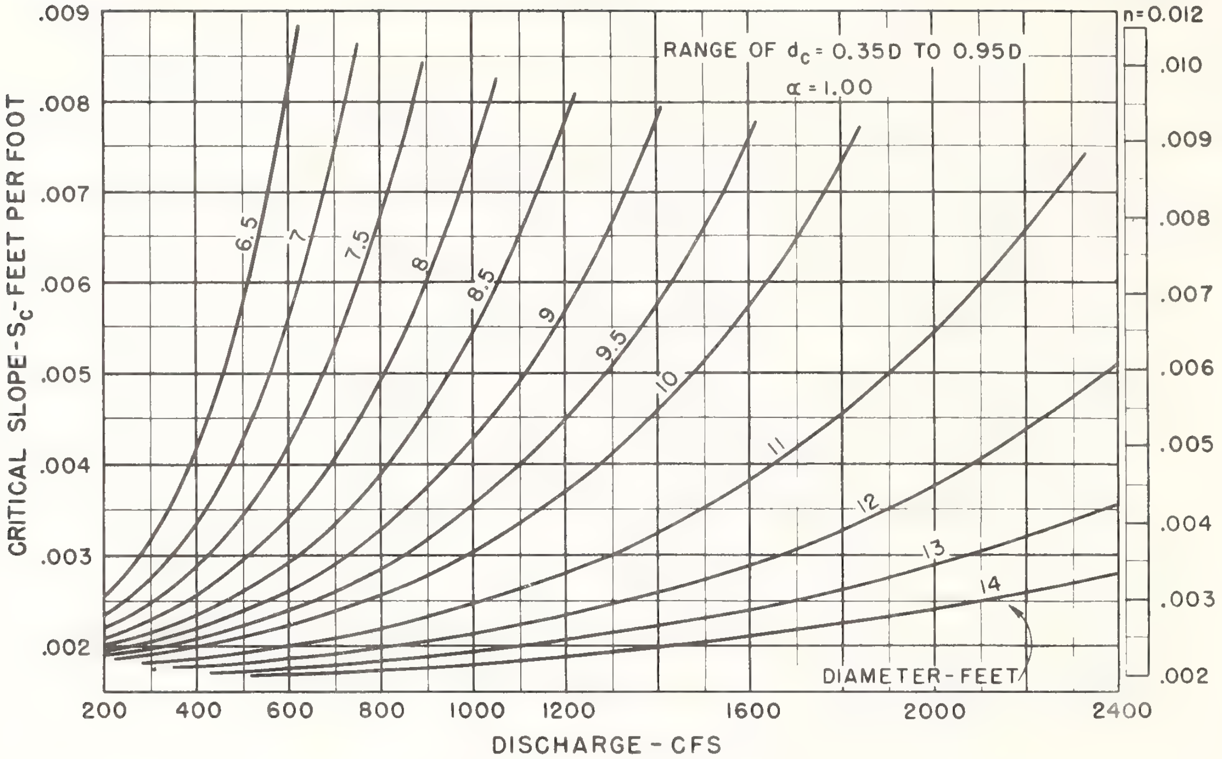


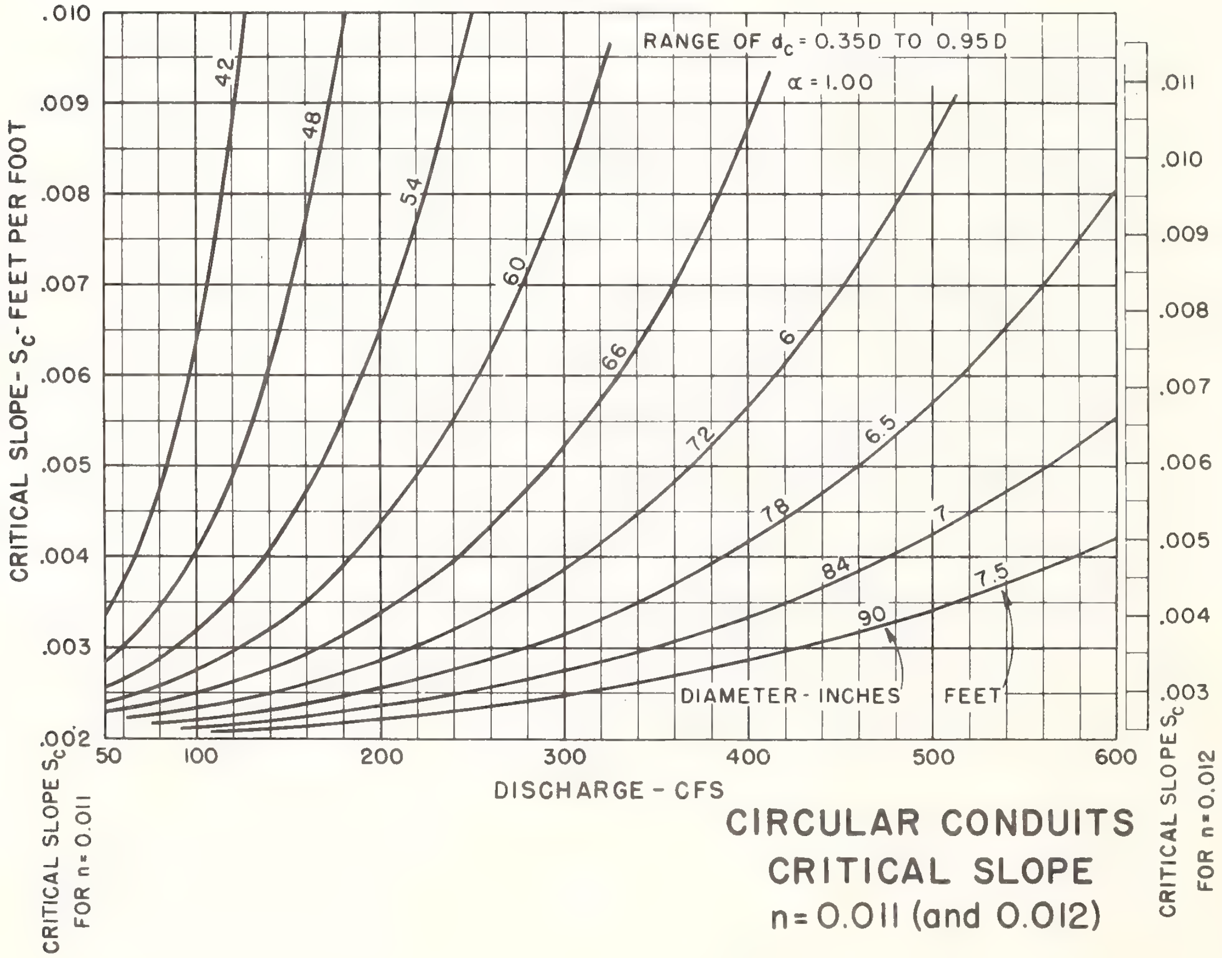


CIRCULAR C. M. PIPE
CRITICAL SLOPE
n = 0.025



CIRCULAR CONDUITS
CRITICAL SLOPE
n=0.011 (and 0.012)



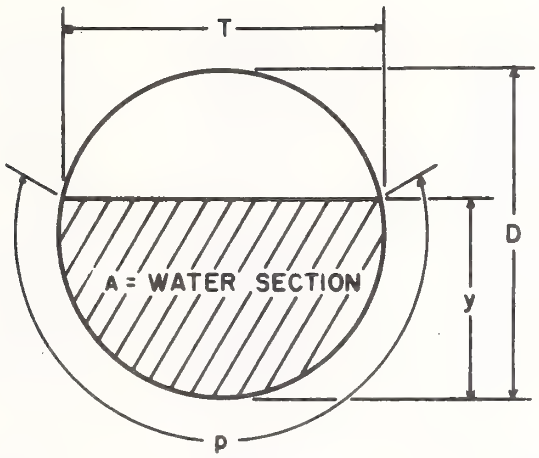


CIRCULAR CONDUITS CRITICAL SLOPE

n = 0.011 (and 0.012)

Table 4.34

Geometric Elements Circular Section Partially Full



$r = \frac{A}{p}$ = HYDRAULIC RADIUS

Q = DISCHARGE, cfs

$\frac{y}{D}$	$\frac{A}{D^2}$	$\frac{P}{D}$	$\frac{r}{D}$	$\frac{T}{D}$	$\frac{Ar^{2/3}}{D^{8/3}}$	$\frac{Q}{g^{1/2}D^{5/2}}$
0.01	0.0013	0.2003	0.0066	0.1990	0.0000	0.0001
0.02	0.0037	0.2838	0.0132	0.2800	0.0002	0.0004
0.03	0.0069	0.3482	0.0197	0.3412	0.0005	0.0010
0.04	0.0105	0.4027	0.0262	0.3919	0.0009	0.0017
0.05	0.0147	0.4510	0.0326	0.4359	0.0015	0.0027
0.06	0.0192	0.4949	0.0389	0.4750	0.0022	0.0039
0.07	0.0242	0.5355	0.0451	0.5103	0.0031	0.0053
0.08	0.0294	0.5735	0.0513	0.5426	0.0040	0.0069
0.09	0.0350	0.6094	0.0574	0.5724	0.0052	0.0087
0.10	0.0409	0.6435	0.0635	0.6000	0.0065	0.0107
0.11	0.0470	0.6761	0.0695	0.6258	0.0079	0.0129
0.12	0.0534	0.7075	0.0754	0.6499	0.0095	0.0153
0.13	0.0600	0.7377	0.0813	0.6726	0.0113	0.0179
0.14	0.0668	0.7670	0.0871	0.6940	0.0131	0.0217
0.15	0.0739	0.7954	0.0929	0.7141	0.0152	0.0238
0.16	0.0811	0.8230	0.0986	0.7332	0.0173	0.0270
0.17	0.0885	0.8500	0.1042	0.7513	0.0196	0.0304
0.18	0.0961	0.8763	0.1097	0.7684	0.0220	0.0339
0.19	0.1039	0.9020	0.1152	0.7846	0.0247	0.0378
0.20	0.1118	0.9273	0.1206	0.8000	0.0273	0.0418
0.21	0.1199	0.9521	0.1259	0.8146	0.0301	0.0460
0.22	0.1281	0.9764	0.1312	0.8285	0.0333	0.0503
0.23	0.1365	0.0003	0.1364	0.8417	0.0359	0.0549
0.24	0.1449	0.0239	0.1416	0.8542	0.0394	0.0597
0.25	0.1535	0.0472	0.1466	0.8660	0.0427	0.0646

Table 4.34 (contd.)

$\frac{y}{D}$	$\frac{A}{D^2}$	$\frac{P}{D}$	$\frac{r}{D}$	$\frac{T}{D}$	$\frac{Ar^{2/3}}{D^{8/3}}$	$\frac{Q}{g^{1/2}D^{5/2}}$
0.26	0.1623	1.0701	0.1516	0.8773	0.0464	0.0697
0.27	0.1711	1.0928	0.1566	0.8879	0.0497	0.0751
0.28	0.1800	1.1152	0.1614	0.8980	0.0536	0.0805
0.29	0.1890	1.1373	0.1662	0.9075	0.0571	0.0862
0.30	0.1982	1.1593	0.1709	0.9165	0.0610	0.0921
0.31	0.2074	1.1810	0.1755	0.9250	0.0650	0.0981
0.32	0.2167	1.2025	0.1801	0.9330	0.0690	0.1044
0.33	0.2260	1.2239	0.1848	0.9404	0.0736	0.1107
0.34	0.2355	1.2451	0.1891	0.9474	0.0776	0.1172
0.35	0.2450	1.2661	0.1935	0.9539	0.0820	0.1241
0.36	0.2546	1.2870	0.1978	0.9600	0.0864	0.1310
0.37	0.2642	1.3078	0.2020	0.9656	0.0909	0.1381
0.38	0.2739	1.3284	0.2061	0.9708	0.0955	0.1453
0.39	0.2836	1.3490	0.2102	0.9755	0.1020	0.1528
0.40	0.2934	1.3694	0.2142	0.9798	0.1050	0.1603
0.41	0.3032	1.3898	0.2181	0.9837	0.1100	0.1682
0.42	0.3132	1.4101	0.2220	0.9871	0.1147	0.1761
0.43	0.3229	1.4303	0.2257	0.9902	0.1196	0.1844
0.44	0.3328	1.4505	0.2294	0.9928	0.1245	0.1927
0.45	0.3428	1.4706	0.2331	0.9950	0.1298	0.2011
0.46	0.3527	1.4907	0.2366	0.9968	0.1348	0.2098
0.47	0.3627	1.5108	0.2400	0.9982	0.1401	0.2186
0.48	0.3727	1.5308	0.2434	0.9992	0.1452	0.2275
0.49	0.3827	1.5508	0.2467	0.9998	0.1505	0.2366
0.50	0.3927	1.5708	0.2500	1.0000	0.1558	0.2459
0.51	0.4027	1.5908	0.2531	0.9998	0.1610	0.2553
0.52	0.4127	1.6108	0.2561	0.9992	0.1664	0.2650
0.53	0.4227	1.6308	0.2591	0.9982	0.1715	0.2748
0.54	0.4327	1.6509	0.2620	0.9968	0.1772	0.2848
0.55	0.4426	1.6710	0.2649	0.9950	0.1825	0.2949
0.56	0.4526	1.6911	0.2676	0.9928	0.1878	0.3051
0.57	0.4625	1.7113	0.2703	0.9902	0.1933	0.3158
0.58	0.4723	1.7315	0.2728	0.9871	0.1987	0.3263
0.59	0.4822	1.7518	0.2753	0.9837	0.2041	0.3373
0.60	0.4920	1.7722	0.2776	0.9798	0.2092	0.3484
0.61	0.5018	1.7926	0.2797	0.9755	0.2146	0.3560
0.62	0.5115	1.8132	0.2818	0.9708	0.2199	0.3710
0.63	0.5212	1.8338	0.2839	0.9656	0.2252	0.3830
0.64	0.5308	1.8546	0.2860	0.9600	0.2302	0.3945
0.65	0.5404	1.8755	0.2881	0.9539	0.2358	0.4066
0.66	0.5499	1.8965	0.2899	0.9474	0.2407	0.4188
0.67	0.5594	1.9177	0.2917	0.9404	0.2460	0.4309
0.68	0.5687	1.9391	0.2935	0.9330	0.2510	0.4437
0.69	0.5780	1.9606	0.2950	0.9250	0.2560	0.4566
0.70	0.5872	1.9823	0.2962	0.9165	0.2608	0.4694

Table 4.34 (contd.)

$\frac{y}{D}$	$\frac{A}{D^2}$	$\frac{P}{D}$	$\frac{r}{D}$	$\frac{T}{D}$	$\frac{Ar^{2/3}}{D^{8/3}}$	$\frac{Q}{g^{1/2}D^{5/2}}$
0.71	0.5964	2.0042	0.2973	0.9075	0.2653	0.4831
0.72	0.6054	2.0264	0.2984	0.8980	0.2702	0.4964
0.73	0.6143	2.0488	0.2995	0.8879	0.2751	0.5100
0.74	0.6231	2.0714	0.3006	0.8773	0.2794	0.5248
0.75	0.6318	2.0944	0.3017	0.8660	0.2840	0.5392
0.76	0.6404	2.1176	0.3025	0.8542	0.2888	0.5540
0.77	0.6489	2.1412	0.3032	0.8417	0.2930	0.5695
0.78	0.6573	2.1652	0.3037	0.8285	0.2969	0.5850
0.79	0.6655	2.1895	0.3040	0.8146	0.3008	0.6011
0.80	0.6736	2.2143	0.3042	0.8000	0.3045	0.6177
0.81	0.6815	2.2395	0.3044	0.7846	0.3082	0.6347
0.82	0.6893	2.2653	0.3043	0.7684	0.3118	0.6524
0.83	0.6969	2.2916	0.3041	0.7513	0.3151	0.6707
0.84	0.7043	2.3186	0.3038	0.7332	0.3182	0.6897
0.85	0.7115	2.3462	0.3033	0.7141	0.3212	0.7098
0.86	0.7186	2.3746	0.3026	0.6940	0.3240	0.7307
0.87	0.7254	2.4038	0.3017	0.6726	0.3264	0.7528
0.88	0.7320	2.4341	0.3008	0.6499	0.3286	0.7754
0.89	0.7380	2.4655	0.2996	0.6258	0.3307	0.8016
0.90	0.7445	2.4981	0.2980	0.6000	0.3324	0.8285
0.91	0.7504	2.5322	0.2963	0.5724	0.3336	0.8586
0.92	0.7560	2.5681	0.2944	0.5426	0.3345	0.8917
0.93	0.7612	2.6061	0.2922	0.5103	0.3350	0.9292
0.94	0.7662	2.6467	0.2896	0.4750	0.3353	0.9725
0.95	0.7707	2.6906	0.2864	0.4359	0.3349	1.0242
0.96	0.7749	2.7389	0.2830	0.3919	0.3340	1.0888
0.97	0.7785	2.7934	0.2787	0.3412	0.3322	1.1752
0.98	0.7816	2.8578	0.2735	0.2800	0.3291	1.3050
0.99	0.7841	2.9412	0.2665	0.1990	0.3248	1.5554
1.00	0.7854	3.1416	0.2500	0.0000	0.3117	∞

4.65 PIPE-ARCH CHANNELS

Description of Charts - Charts 4.93 - 4.112 are designed for use in the solution of the Manning equation for pipe-arch channels which have sufficient length, on constant slope, to establish uniform flow at normal depth without backwater or pressure head. It is important to recognize that they are not suitable for use in connection with most types of culvert flow, since culvert flow is seldom uniform. The charts are in four groups:

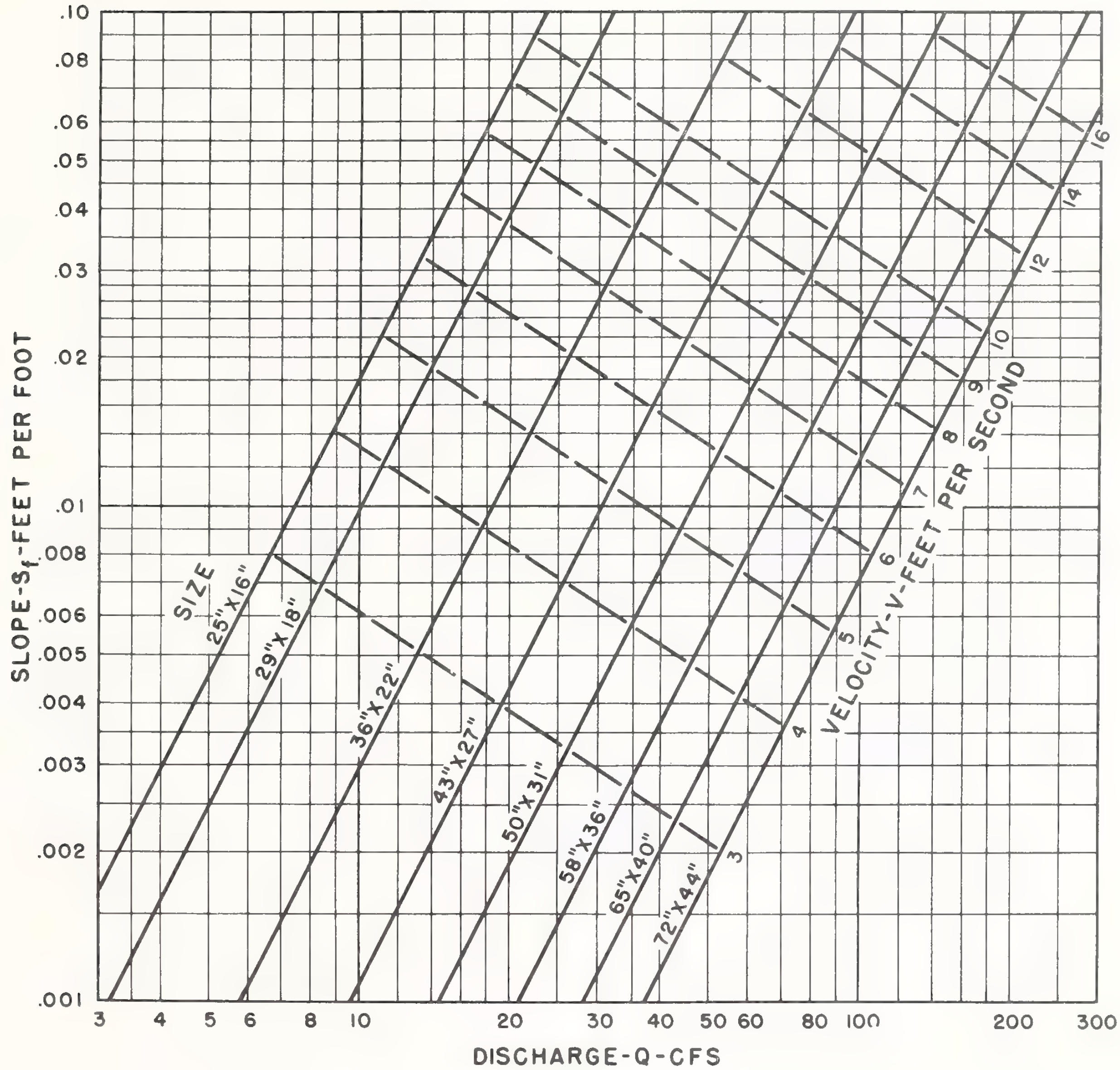
Group 1. - Charts 4.93 - 4.97 are for standard sizes of riveted, corrugated-metal pipe-arches (n=0.024) varying from 25 by 16 inches to 72 by 44 inches in cross section.

Group 2. - Charts 4.98 - 4.100 are for the same sizes of pipe-arches as those in group 1, but group 2 pipe-arches have 40-percent paved inverts (n=0.019). Charts 4.95 - 4.96 of group 1 are also used with group 2 charts to compute critical depth and specific head at critical depth.

Group 3. - Charts 4.101 - 4.105 are for standard sizes of field-bolted corrugated-metal pipe-arches (n=0.025) ranging in cross section from 6 feet, 1 inch, by 4 feet, 7 inches, to 16 feet, 7 inches, by 10 feet, 1 inch.

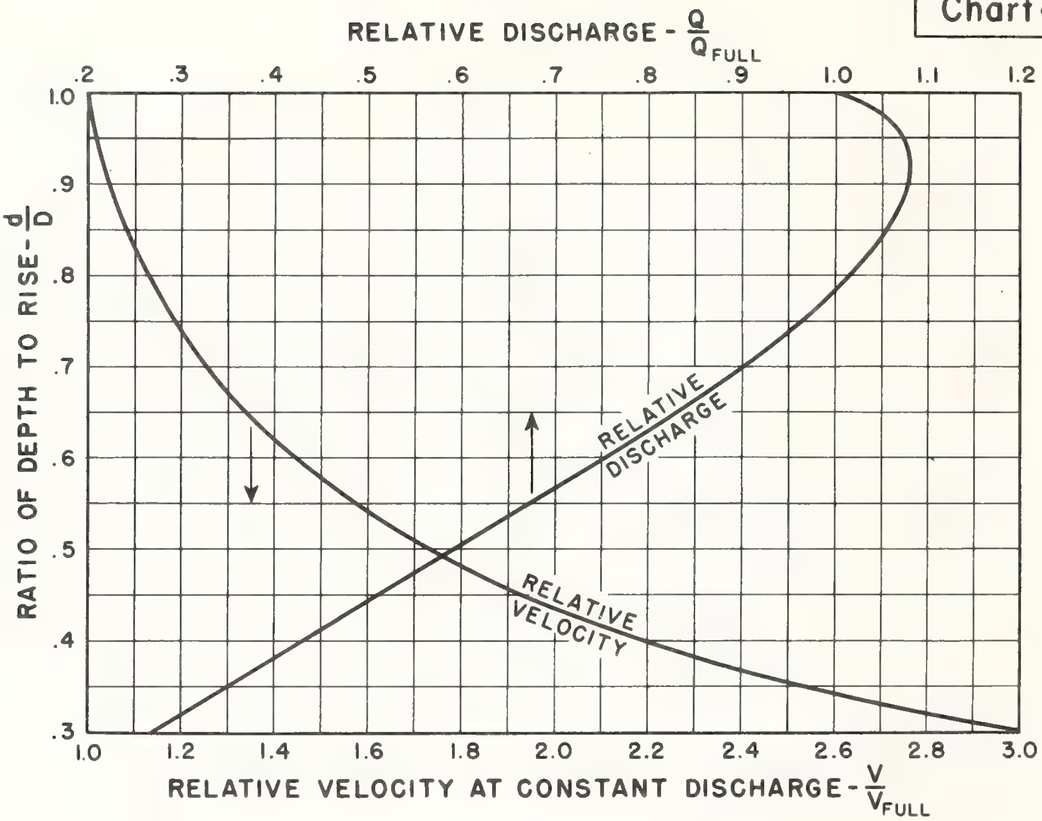
These charts require the use of several charts for solving the Manning equation. Each group of charts consists of a chart showing friction slope, discharge, and velocity for full flow; a chart of ratios for computing partfull flow; and charts for computing critical flow.

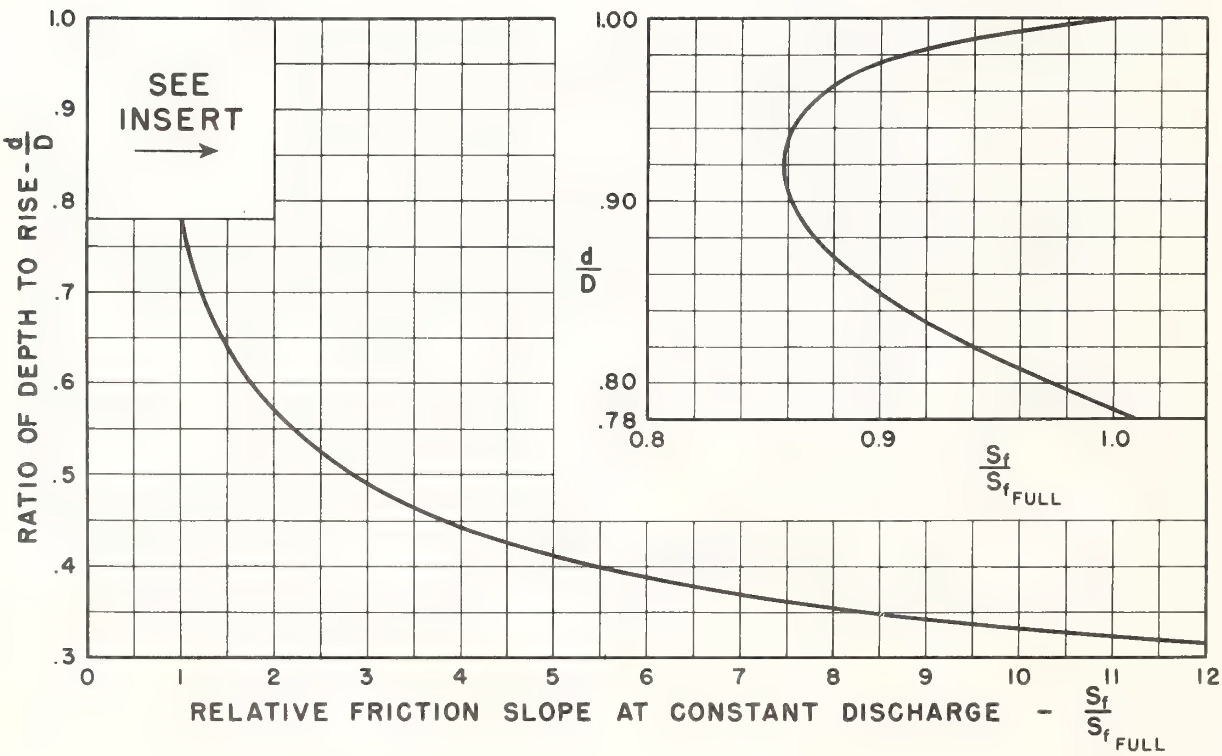
Group 4. - Charts 4.106 - 4.112 are for standard sizes of reinforced concrete pipe arches (n=0.010, 0.011, 0.012, and 0.013) varying from 11 by 18 inches to 87 1/2 by 138 inches.



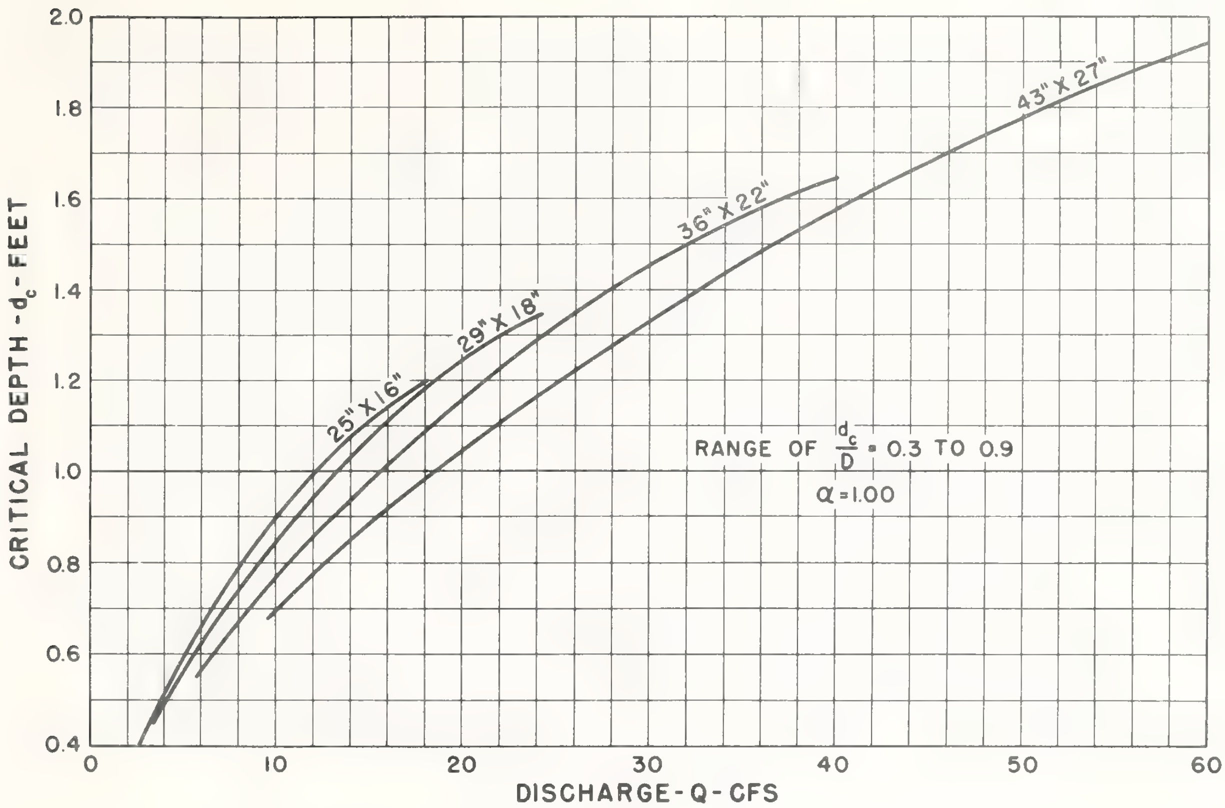
RIVETED C.M. PIPE-ARCH
FRICTION SLOPE FLOWING FULL
n = 0.024

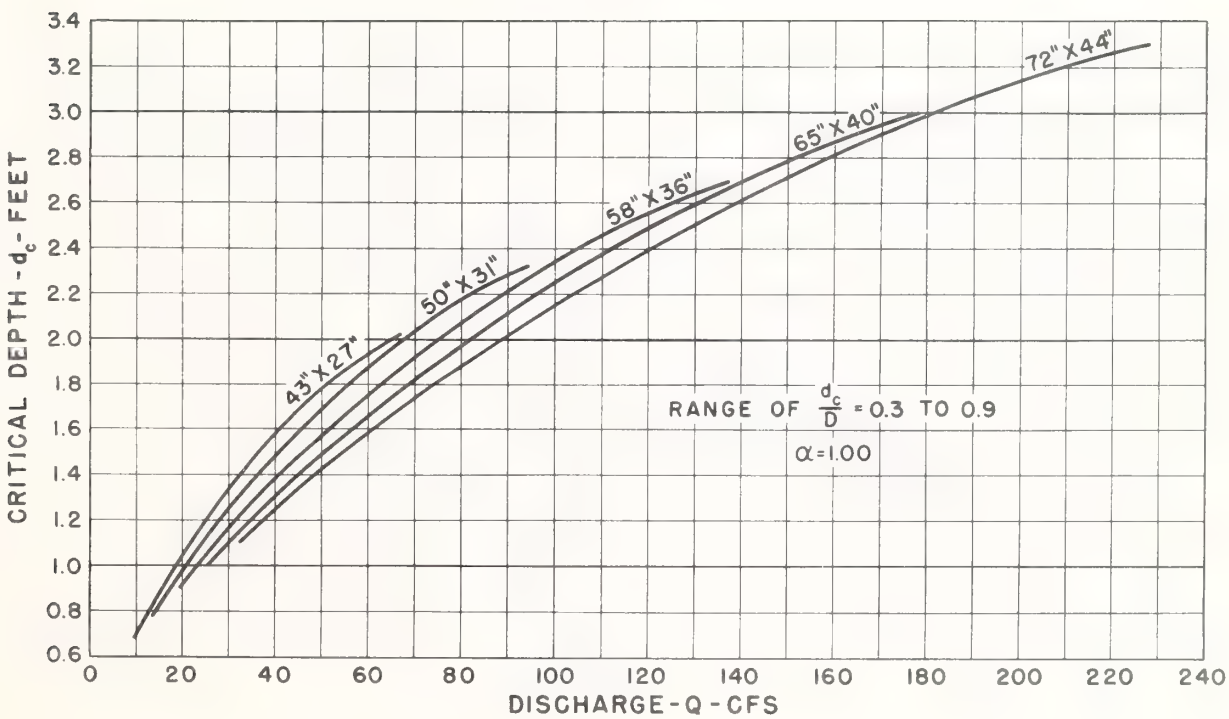
Chart 4.94



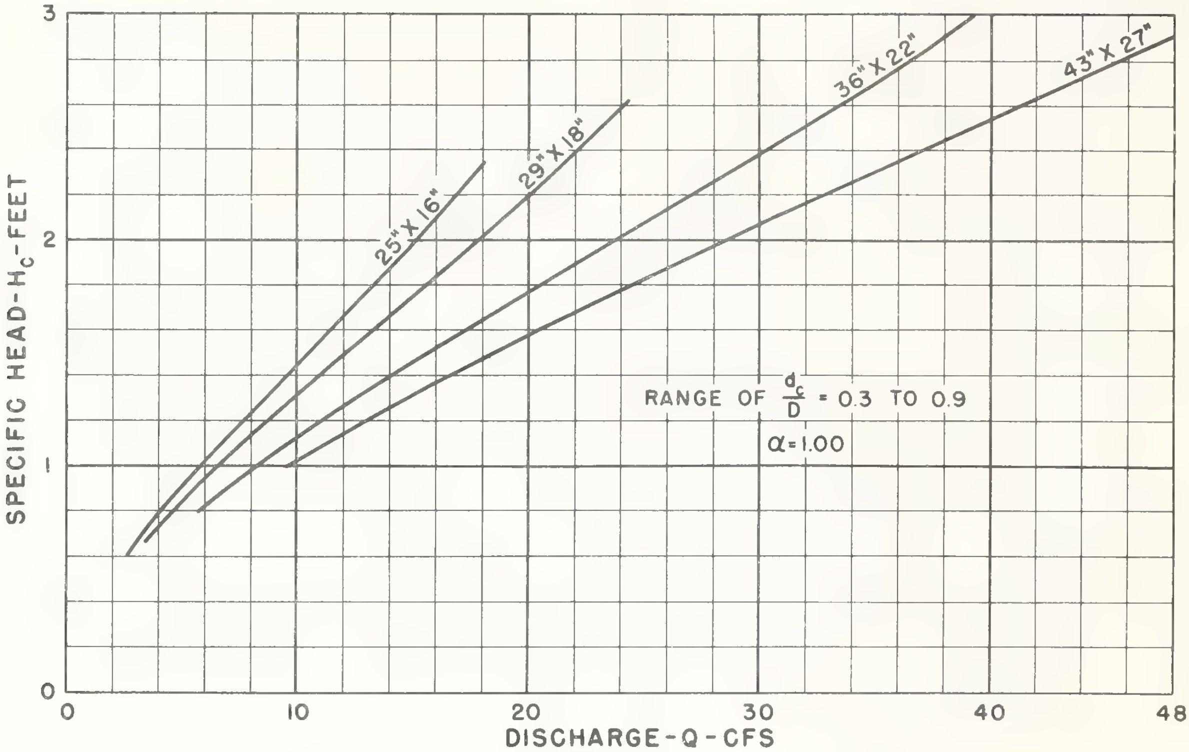


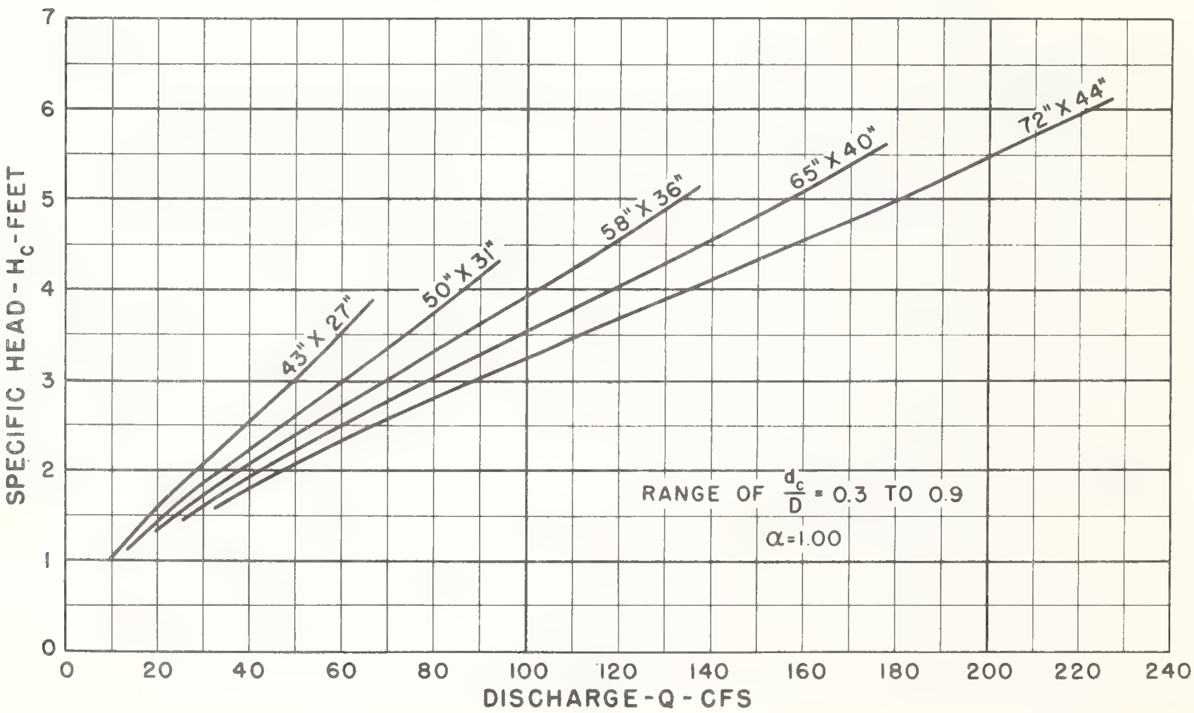
RIVETED C.M. PIPE-ARCH
PART FULL FLOW



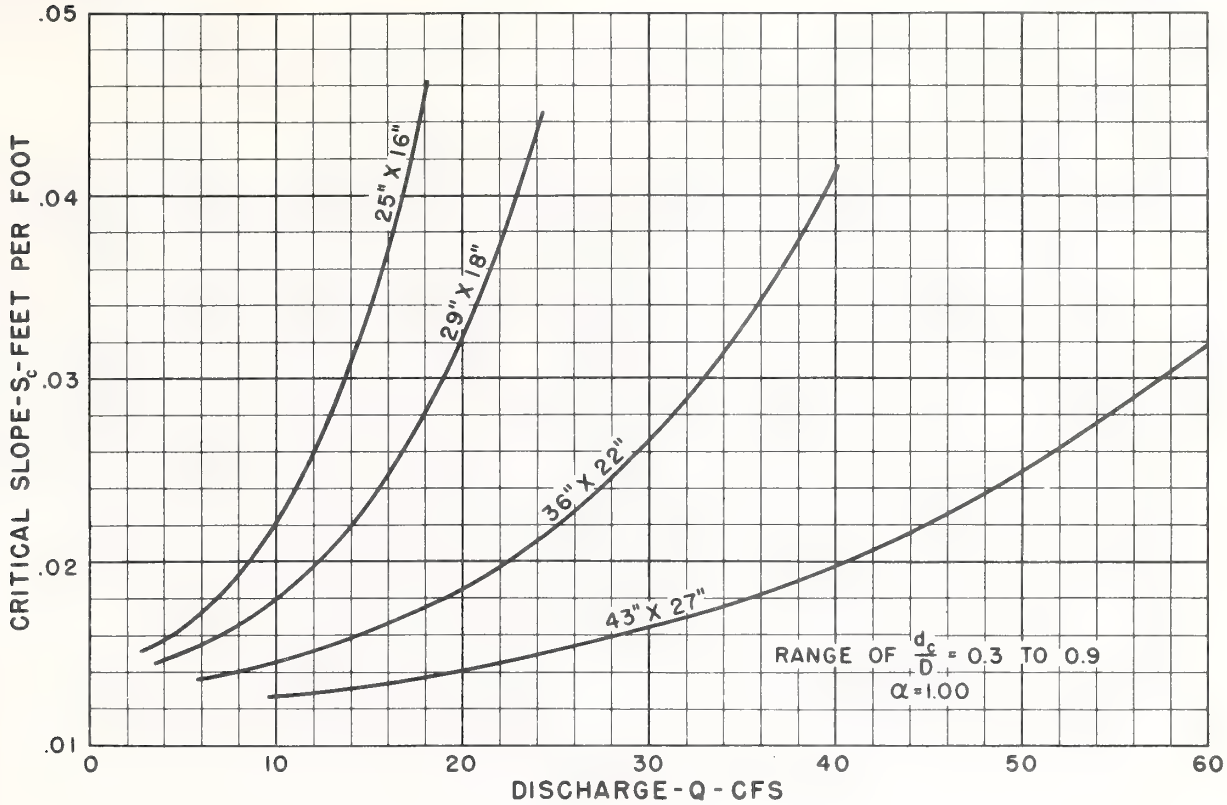


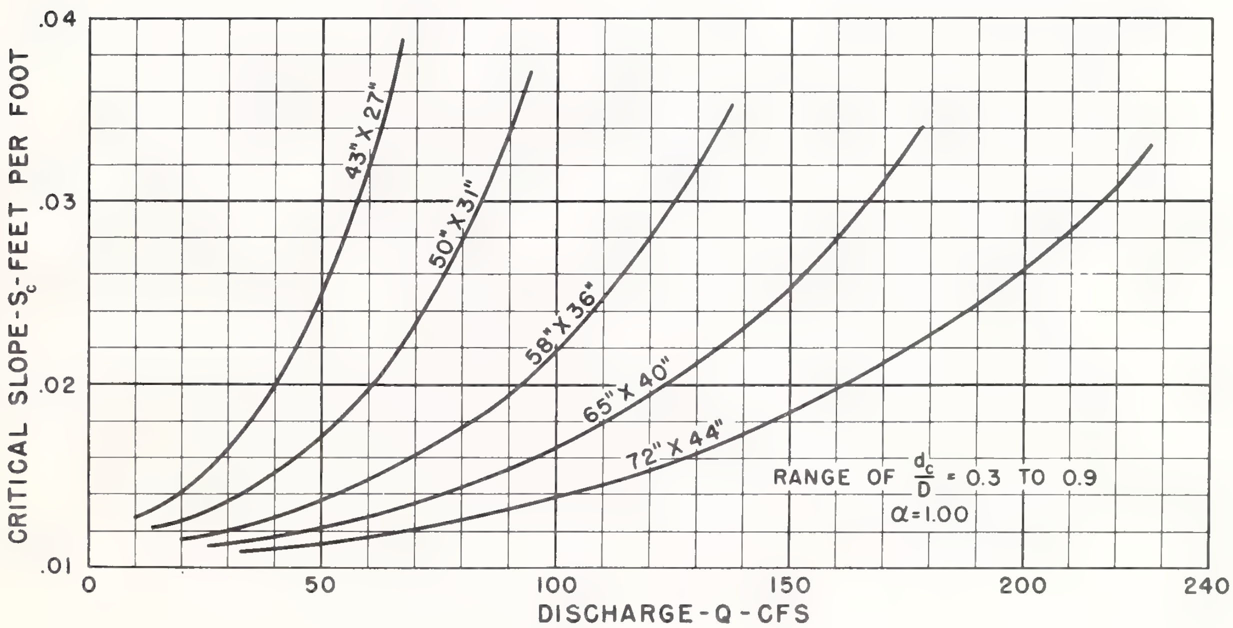
RIVETED C.M. PIPE-ARCH
CRITICAL DEPTH



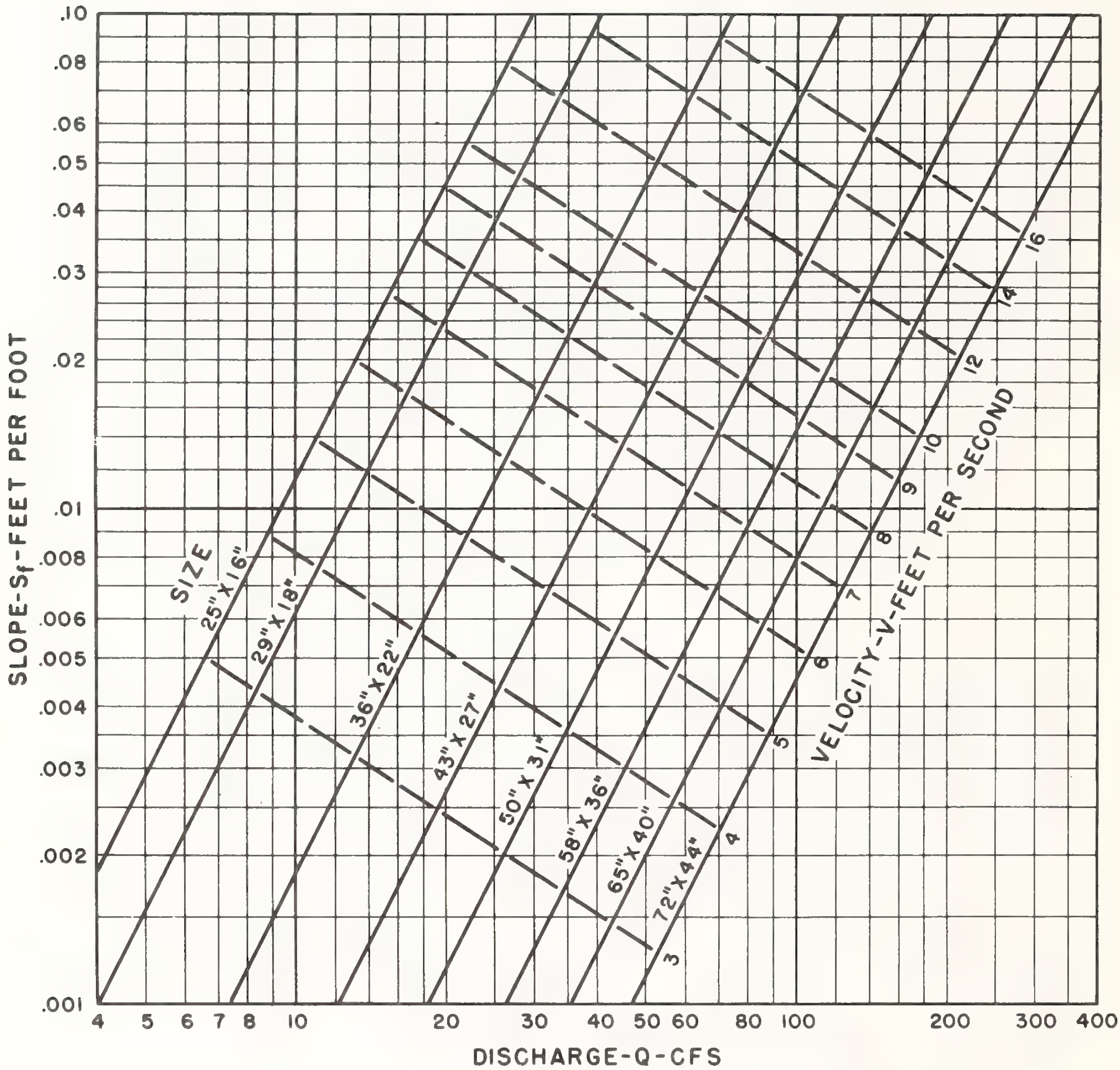


RIVETED C.M. PIPE-ARCH
SPECIFIC HEAD
AT CRITICAL DEPTH

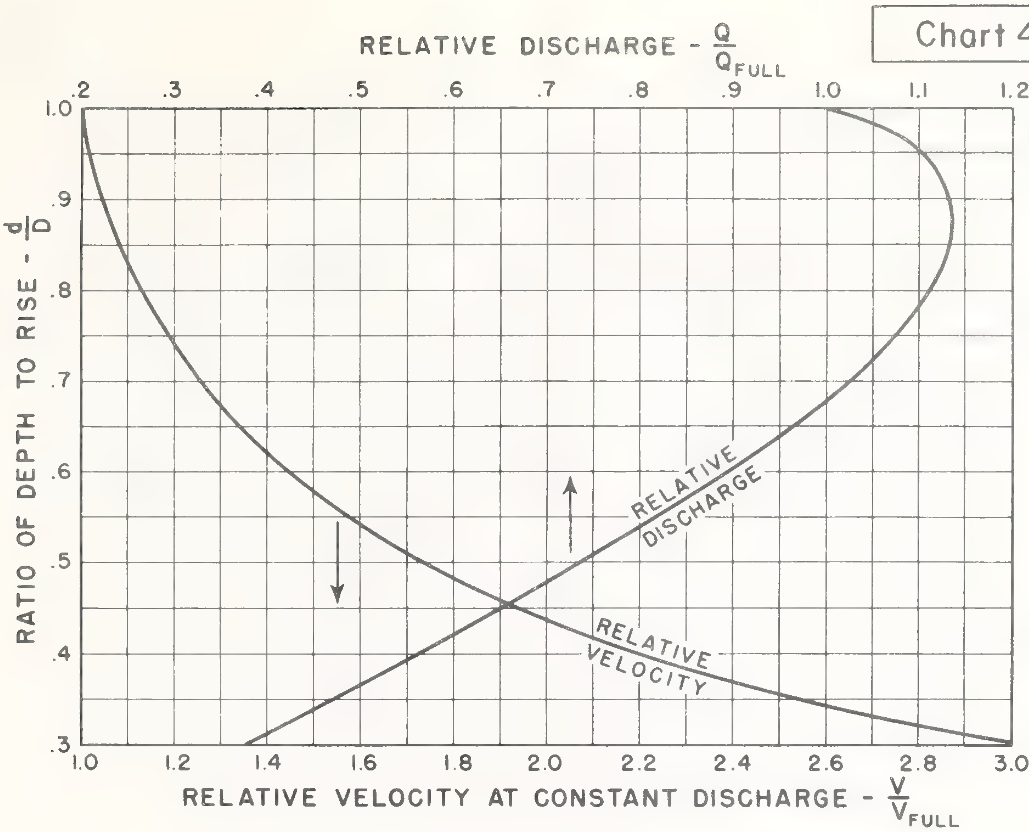


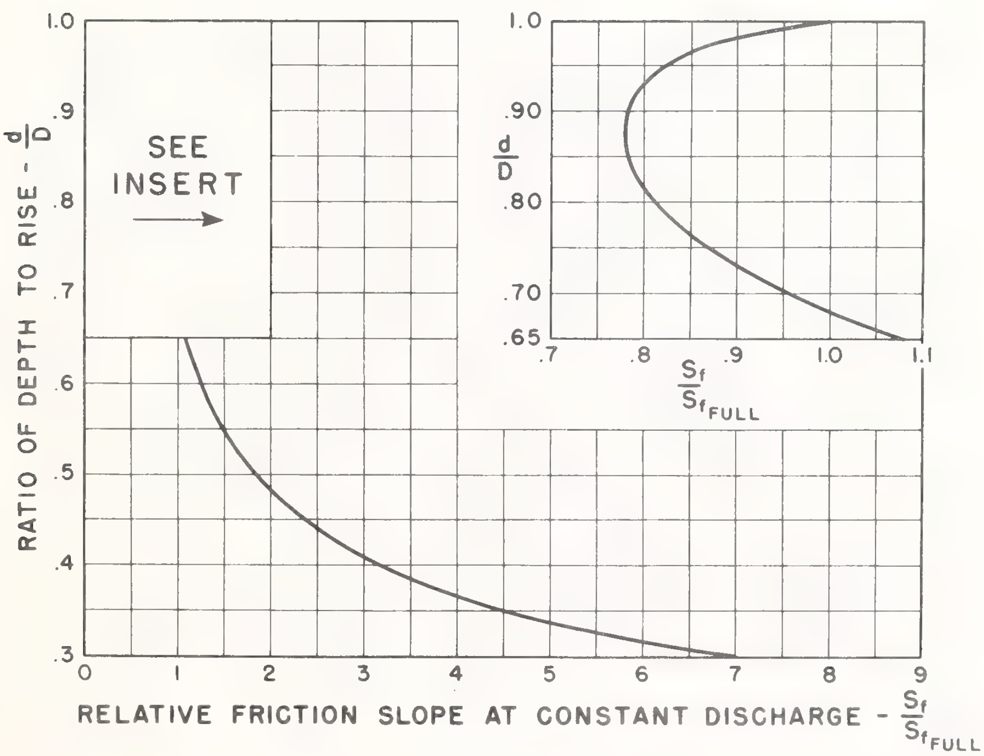


RIVETED C.M. PIPE-ARCH
CRITICAL SLOPE
n = 0.024

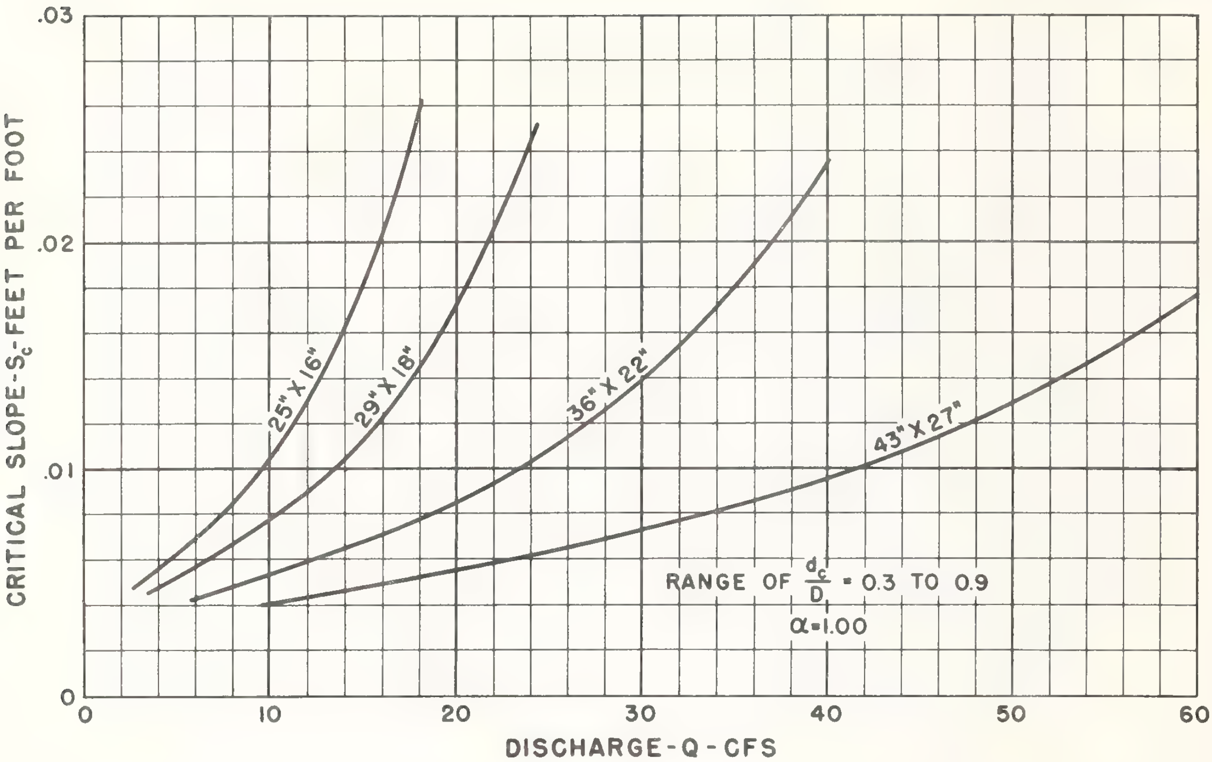


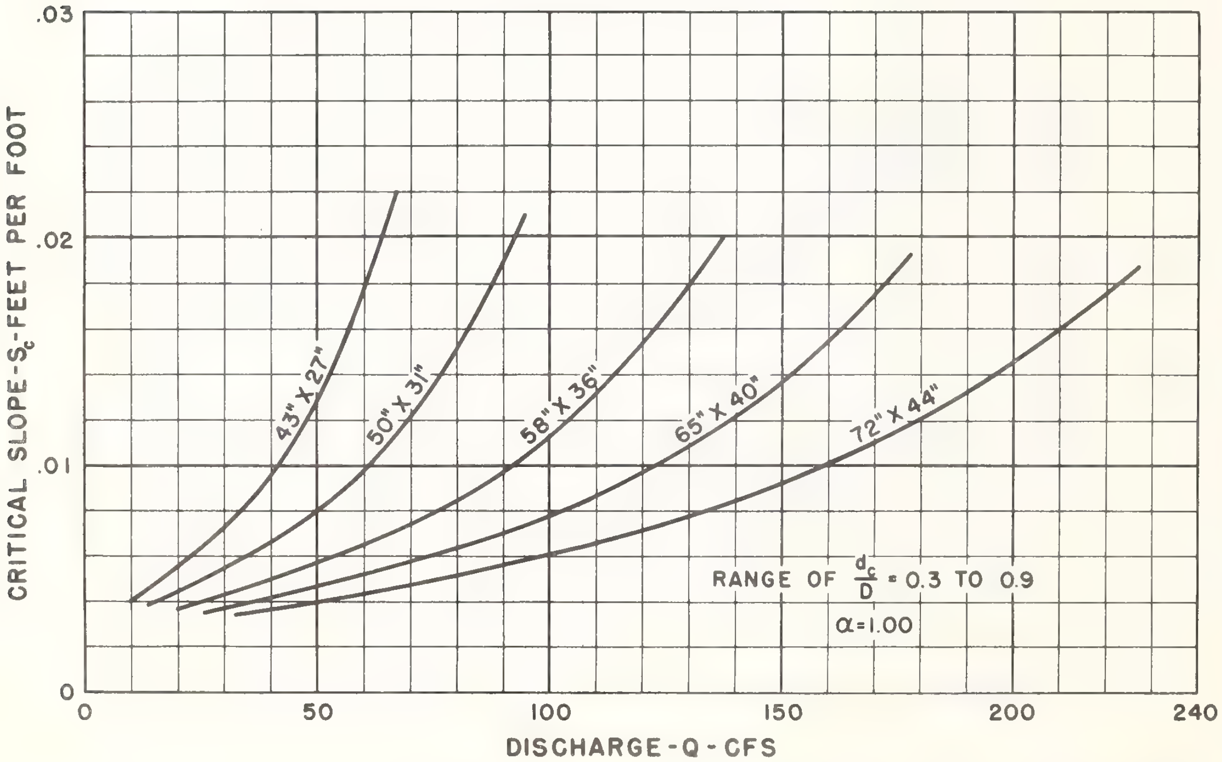
RIVETED C.M. PIPE-ARCH
40% PAVED INVERT
FRICTION SLOPE FLOWING FULL
n=0.019





RIVETED C.M. PIPE-ARCH
40% PAVED INVERT
PART FULL FLOW
n = 0.012 TO 0.019





RIVETED C.M. PIPE-ARCH
40% PAVED INVERT
CRITICAL SLOPE
n = 0.012 TO 0.019

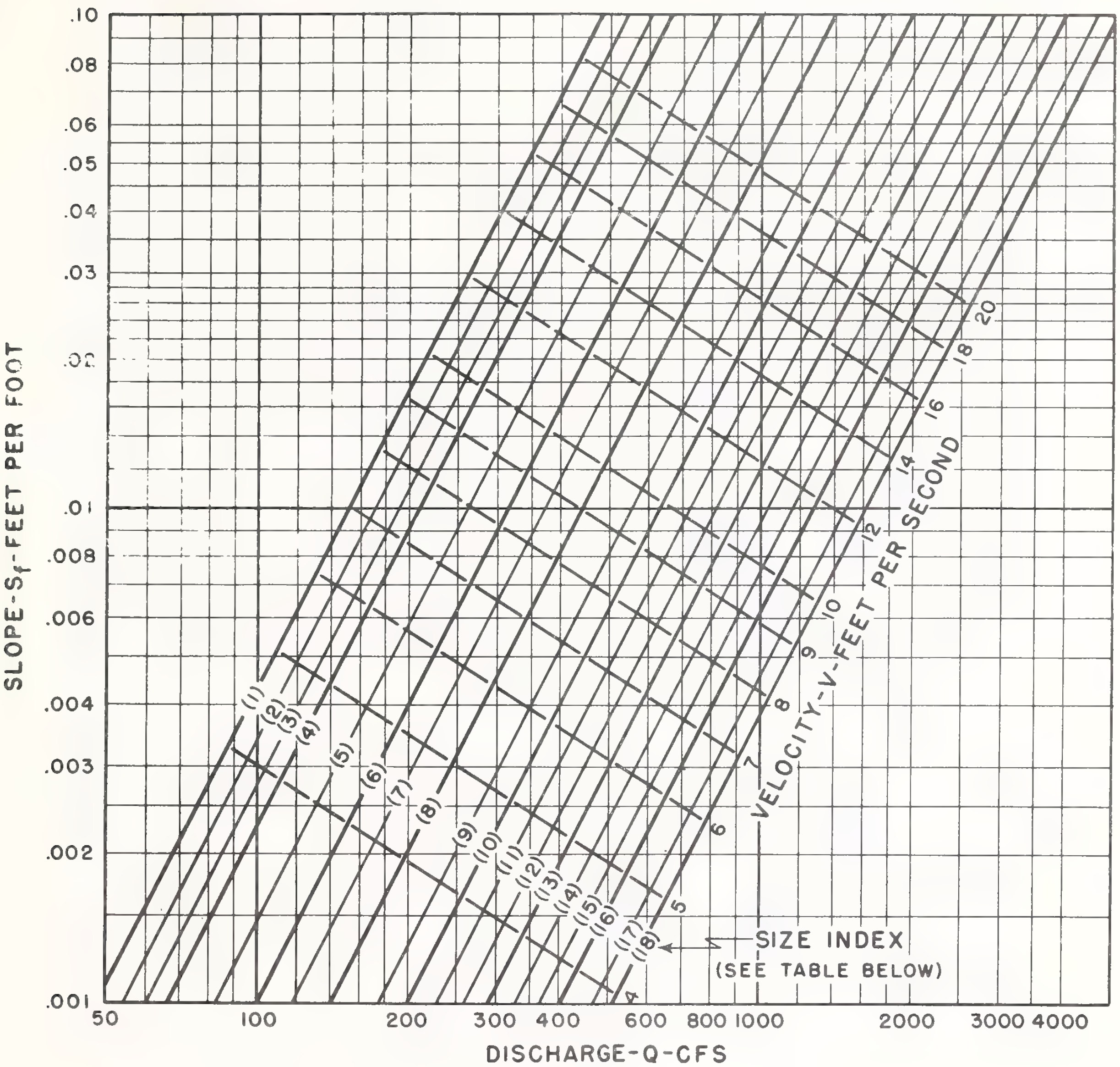
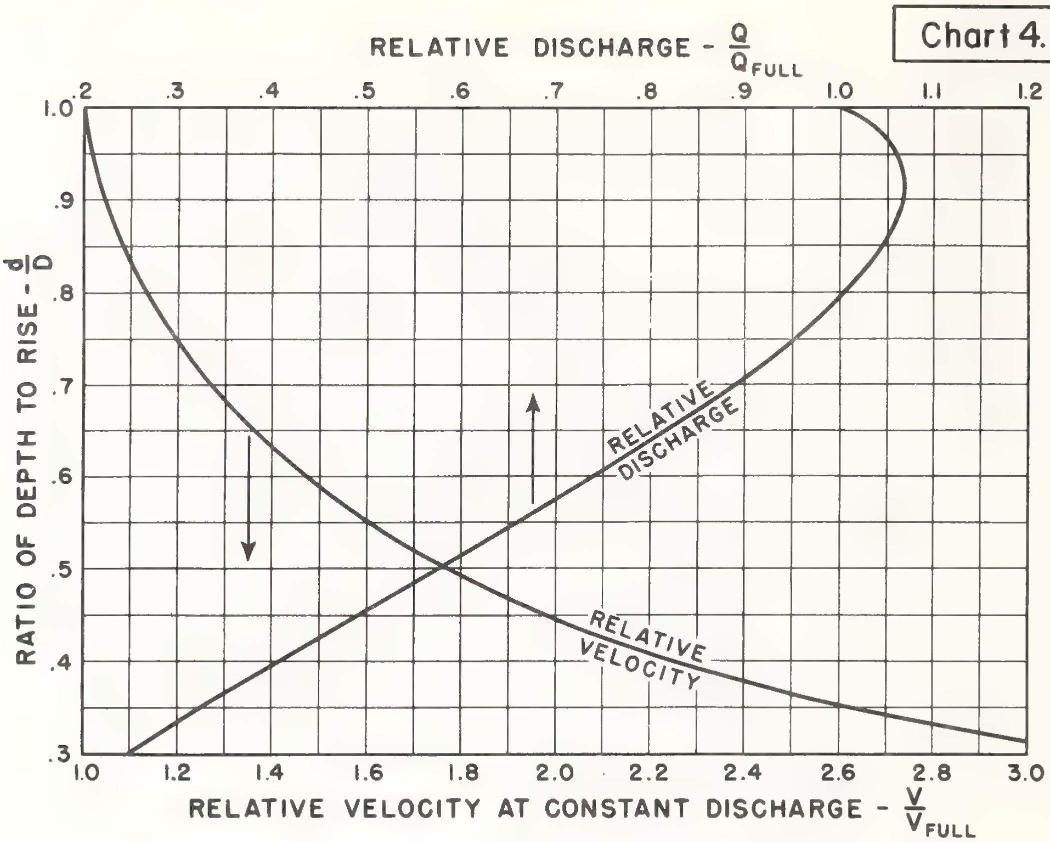
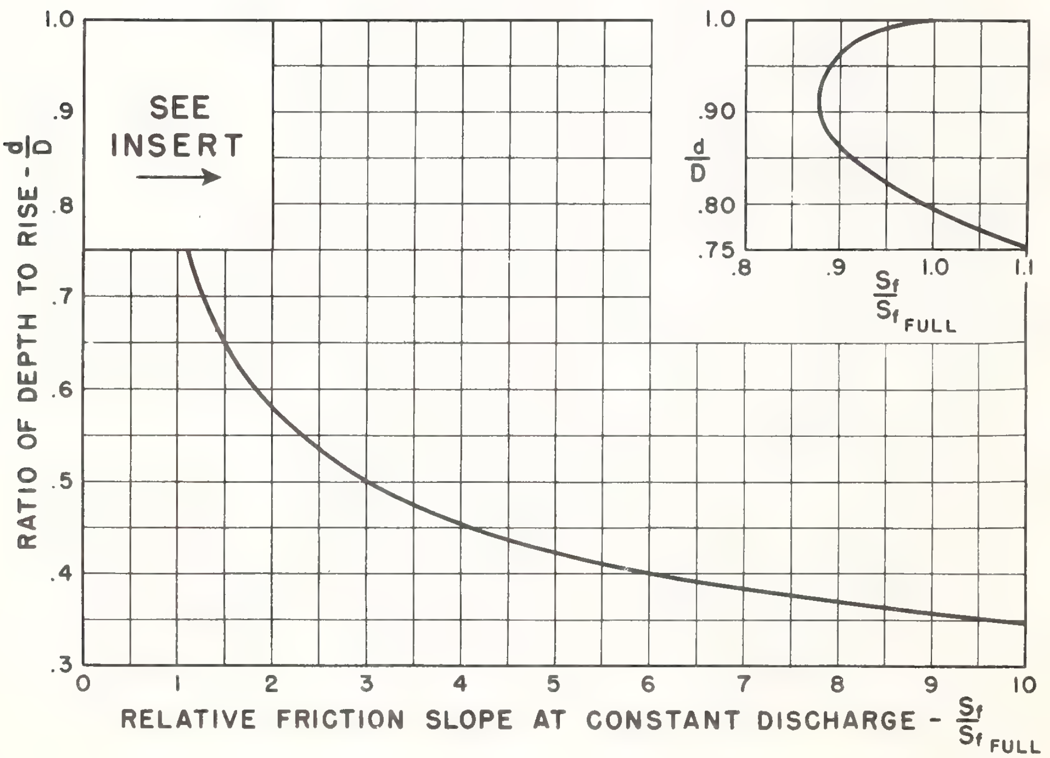


TABLE OF SIZES

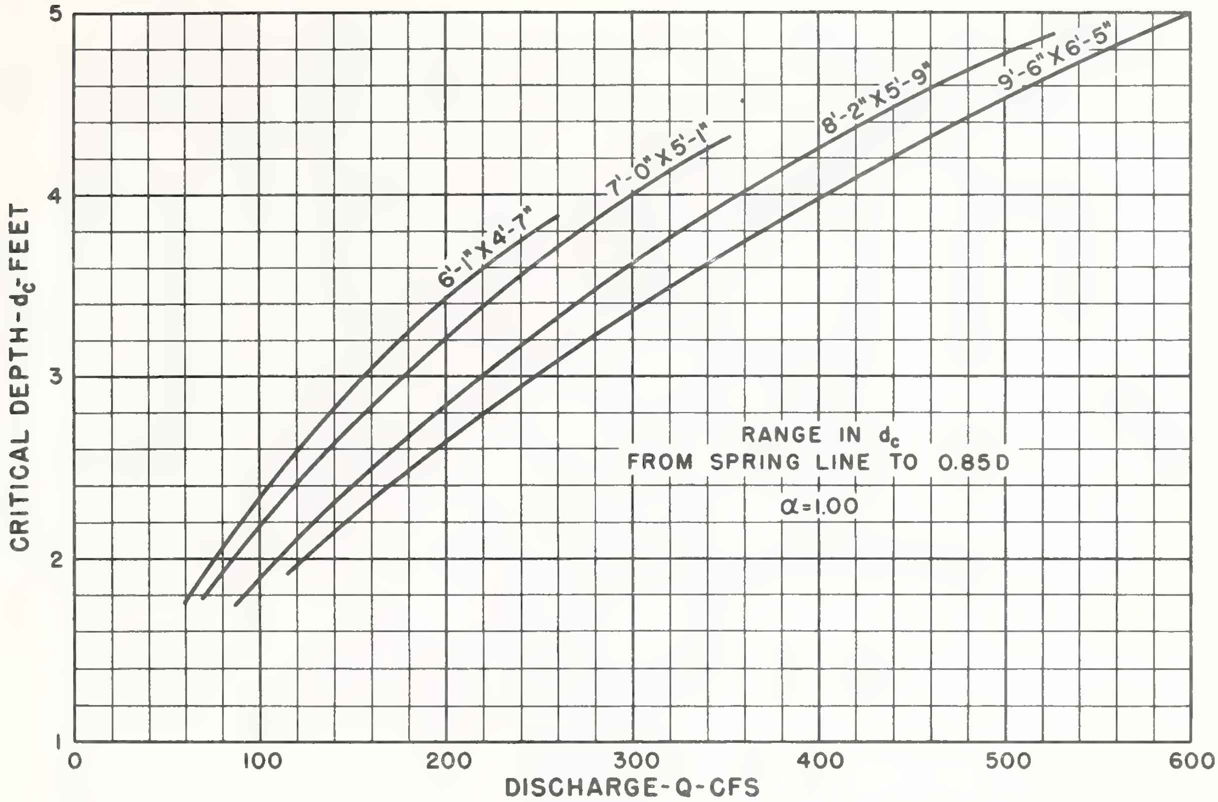
(1) 6'- 1" X 4'- 7"	(7) 8'-10" X 6 - 1"	(13) 12'-10" X 8'- 4"
(2) 6'- 4" X 4'- 9"	(8) 9'- 6" X 6'- 5"	(14) 13'-11" X 8'- 7"
(3) 6'- 9" X 4'-11"	(9) 10'- 8" X 6'-11"	(15) 14'- 3" X 8'- 11"
(4) 7'- 0" X 5'- 1"	(10) 11'- 5" X 7'- 3"	(16) 15'- 4" X 9'- 3"
(5) 7'- 8" X 5'- 5"	(11) 11'-10" X 7'- 7"	(17) 15'-10" X 9'-10"
(6) 8'- 2" X 5'- 9"	(12) 12'- 6" X 7'-11"	(18) 16'- 7" X 10- 1"

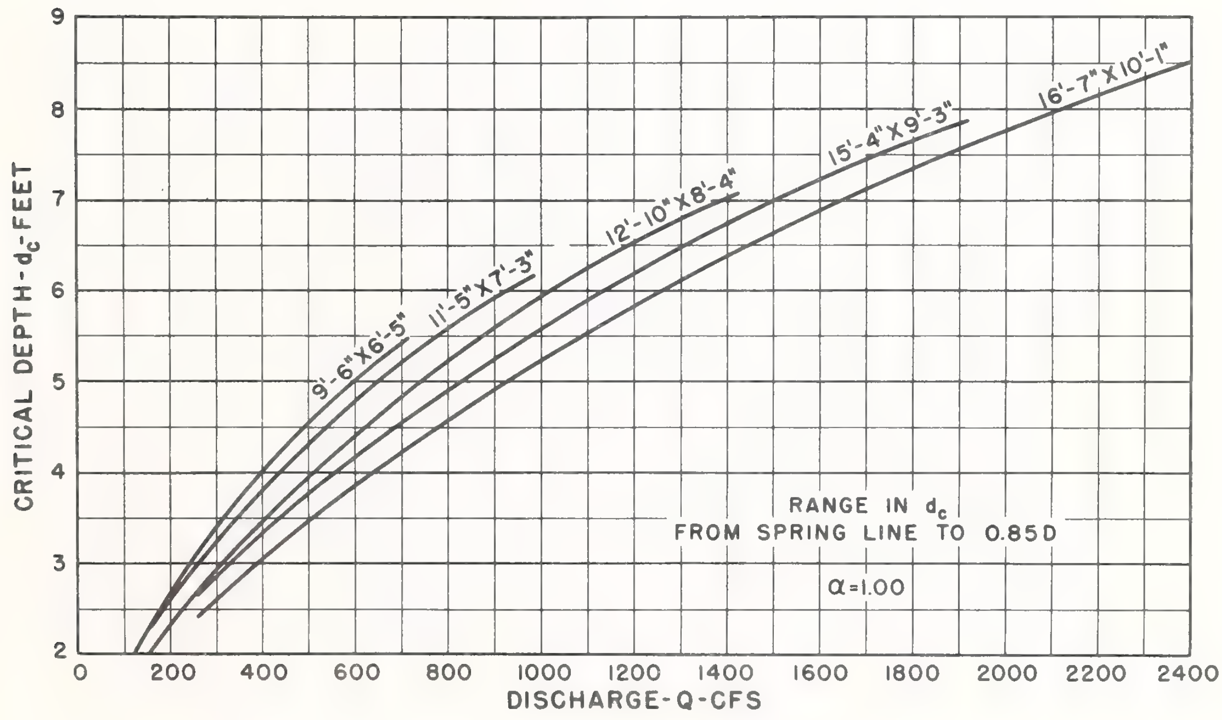
FIELD BOLTED C.M. PIPE-ARCH FRICTION SLOPE FLOWING FULL
n=0.025



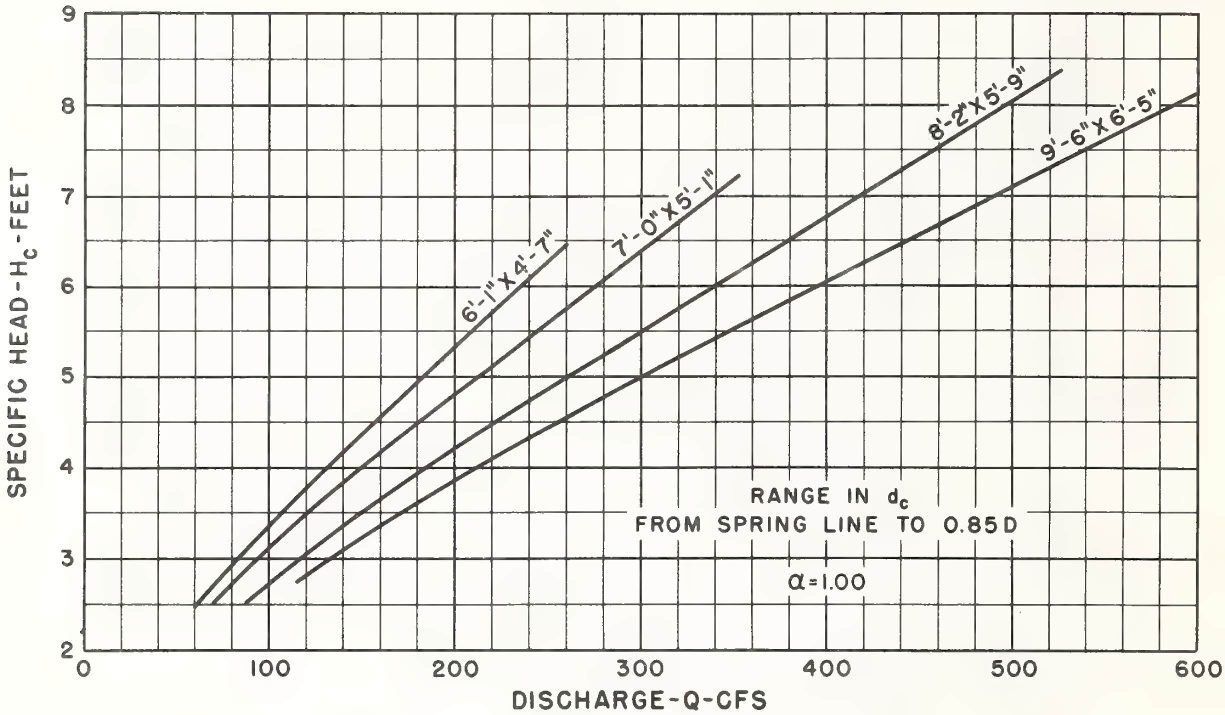


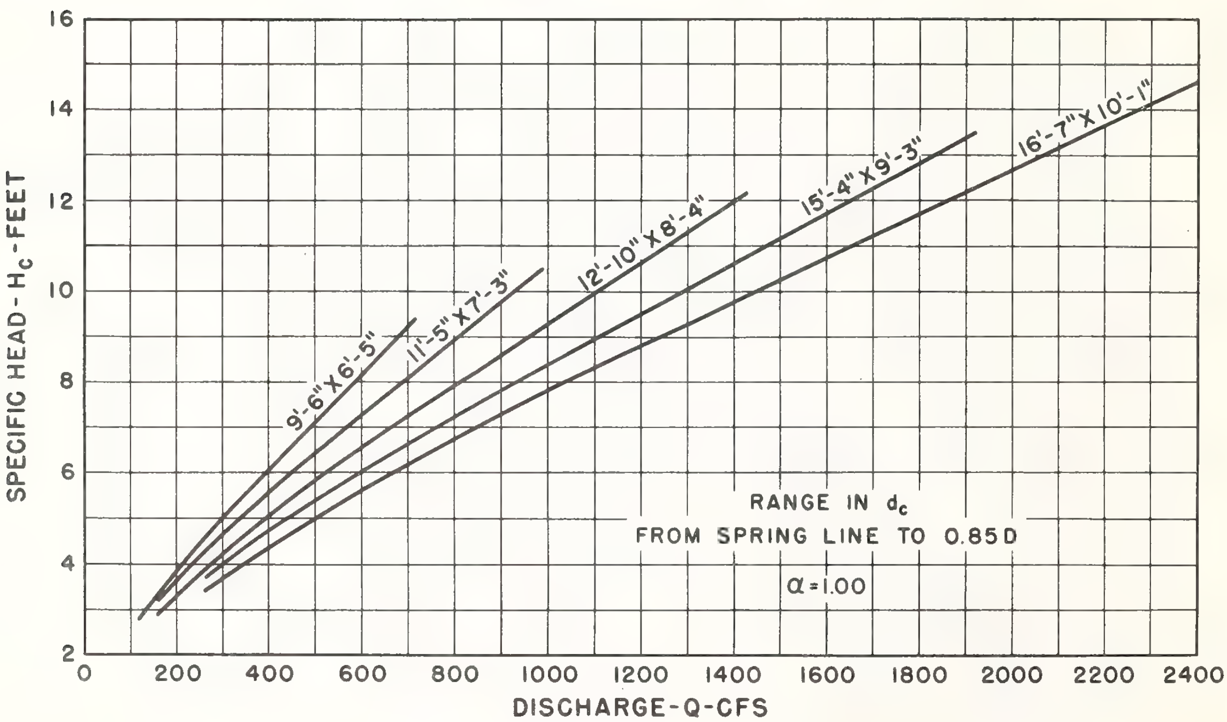
FIELD BOLTED C. M. PIPE-ARCH
PART FULL FLOW





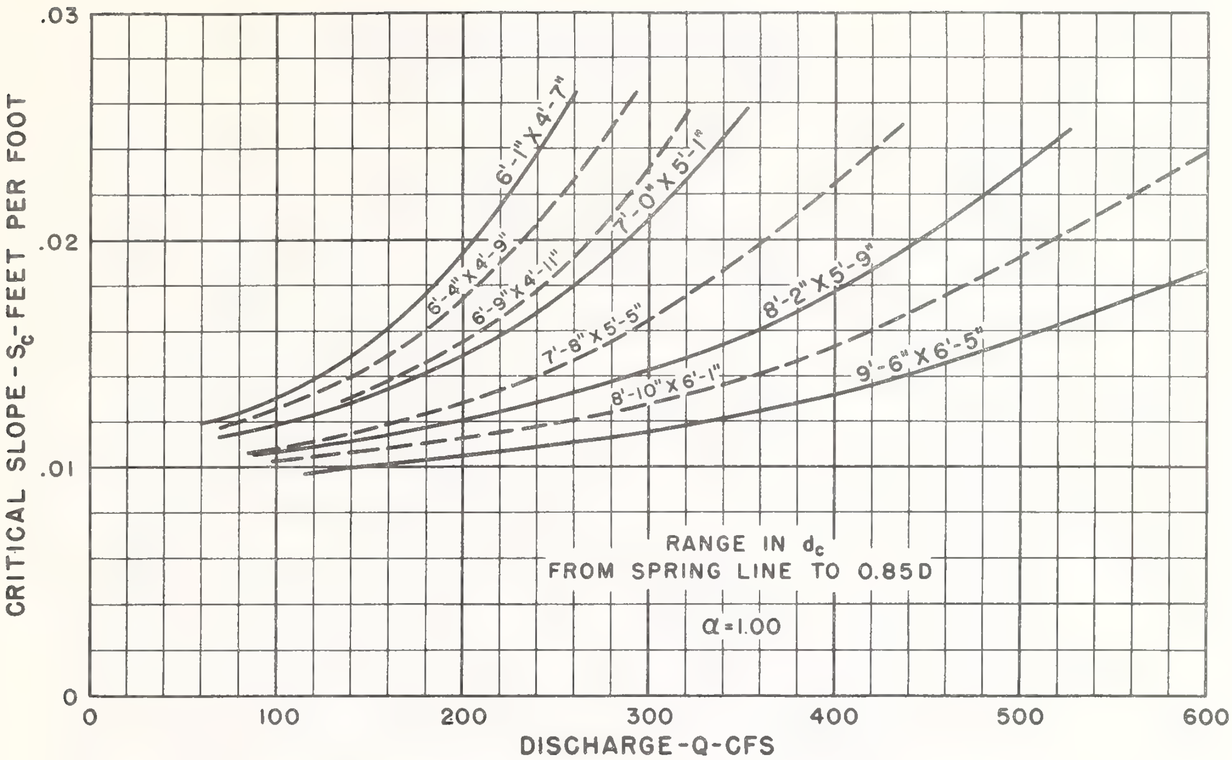
FIELD BOLTED C.M. PIPE-ARCH CRITICAL DEPTH

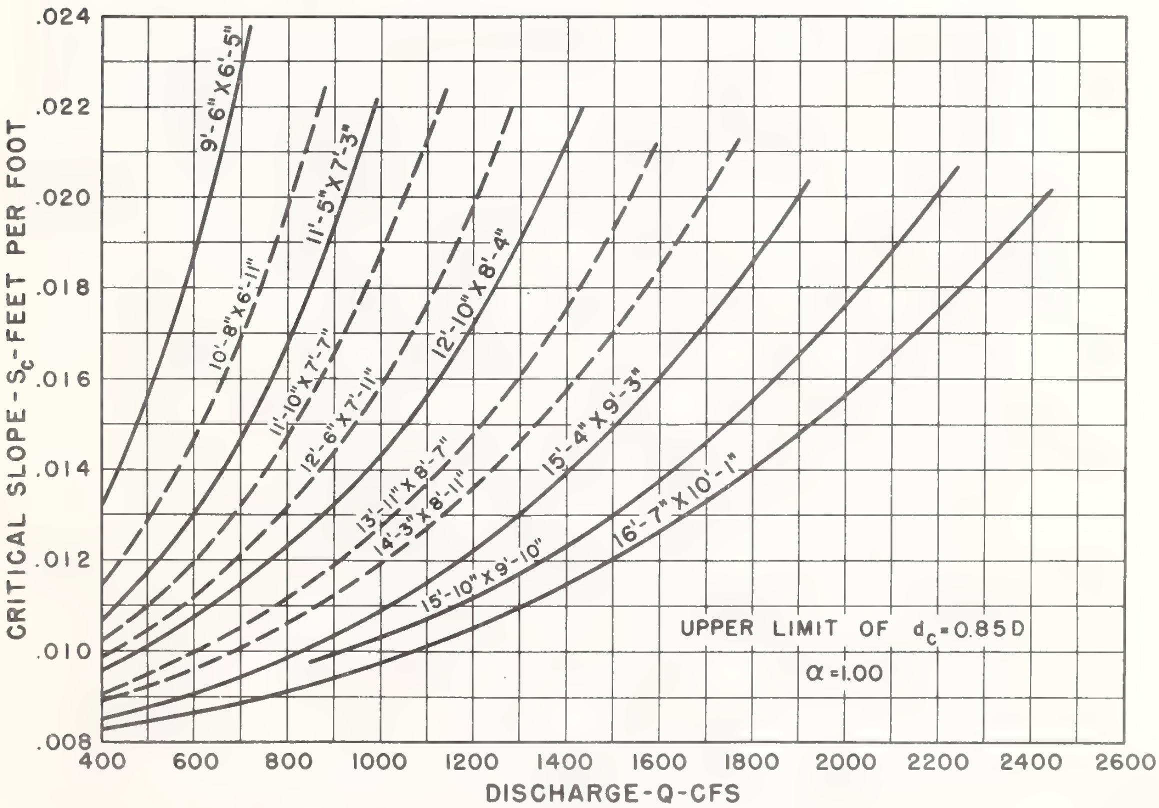




FIELD BOLTED C.M. PIPE-ARCH
SPECIFIC HEAD
AT CRITICAL DEPTH

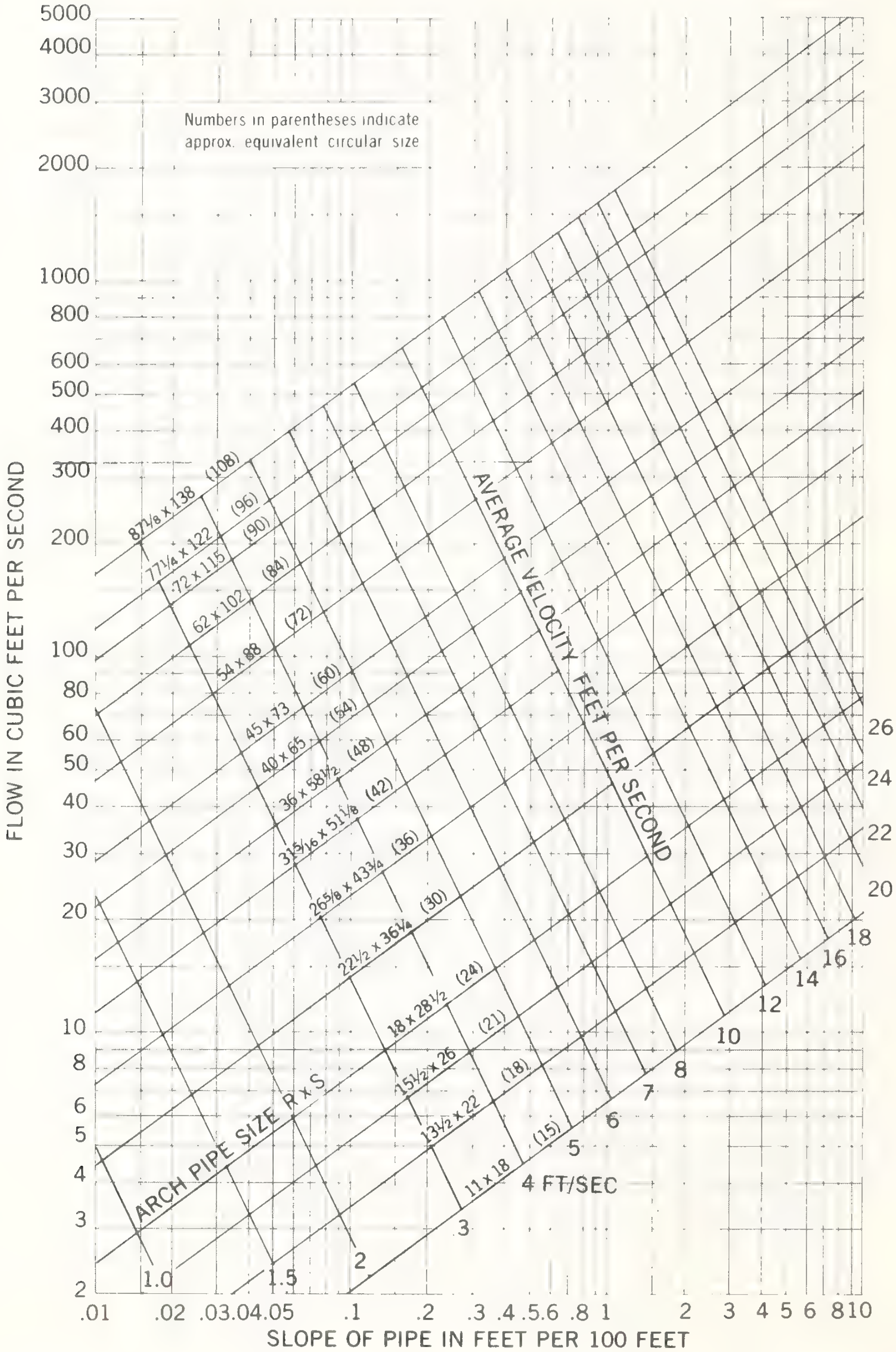
Chart 4.105



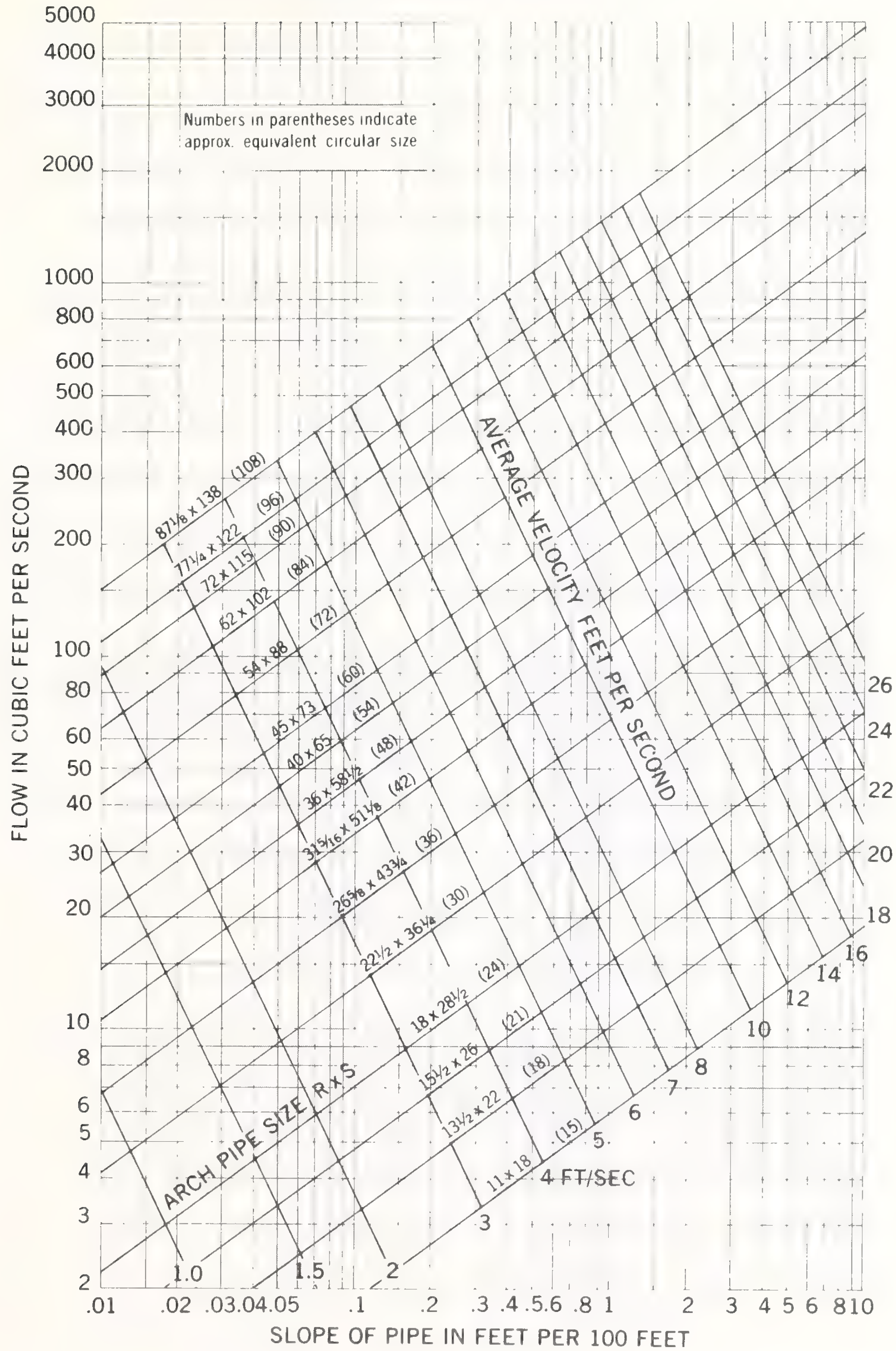


FIELD BOLTED C.M. PIPE-ARCH
CRITICAL SLOPE
n = 0.025

FLOW FOR ARCH PIPE FLOWING FULL
BASED ON MANNING'S EQUATION n=0.010



FLOW FOR ARCH PIPE FLOWING FULL
BASED ON MANNING'S EQUATION n=0.011



FLOW FOR ARCH PIPE FLOWING FULL
BASED ON MANNING'S EQUATION n=0.012

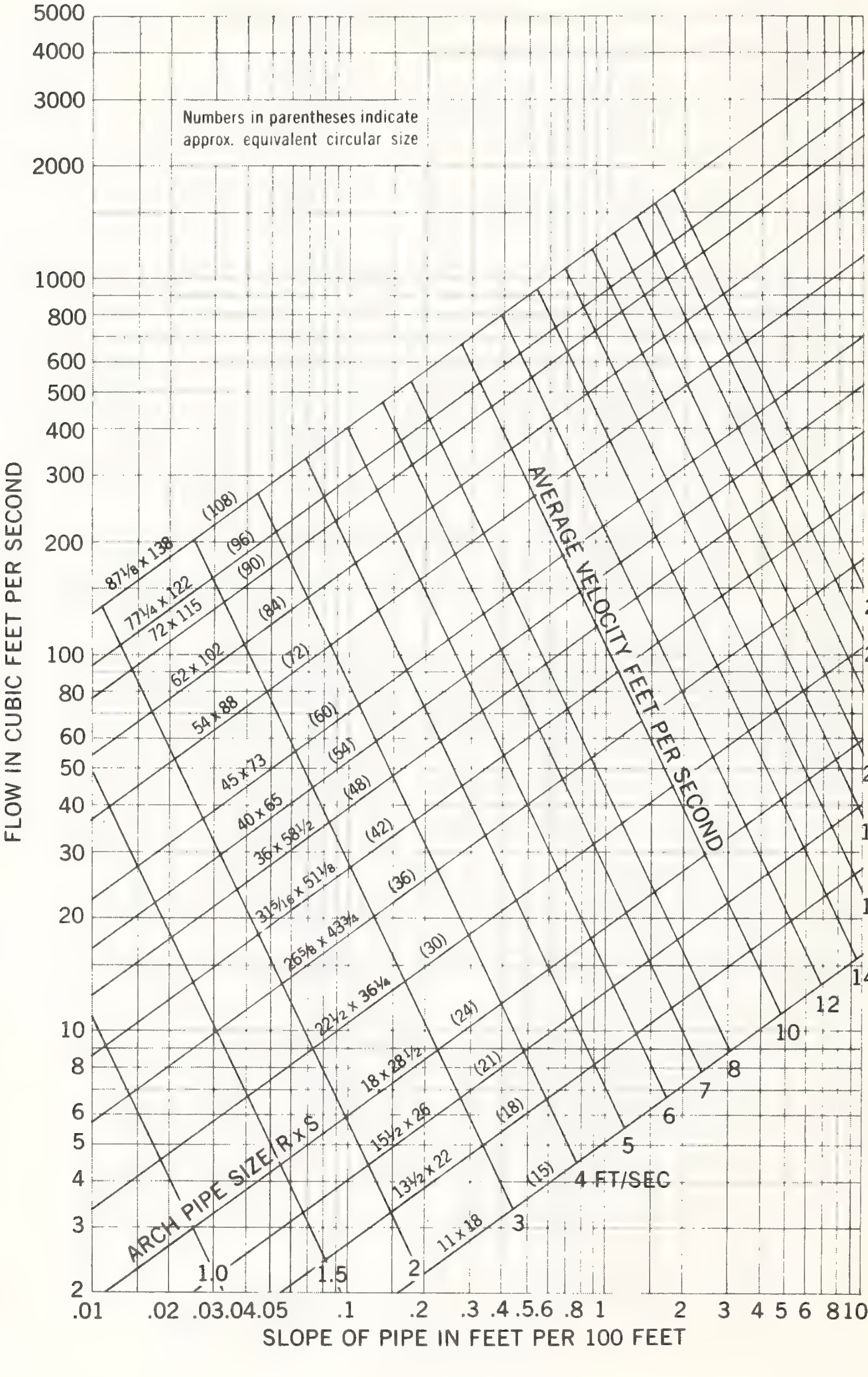
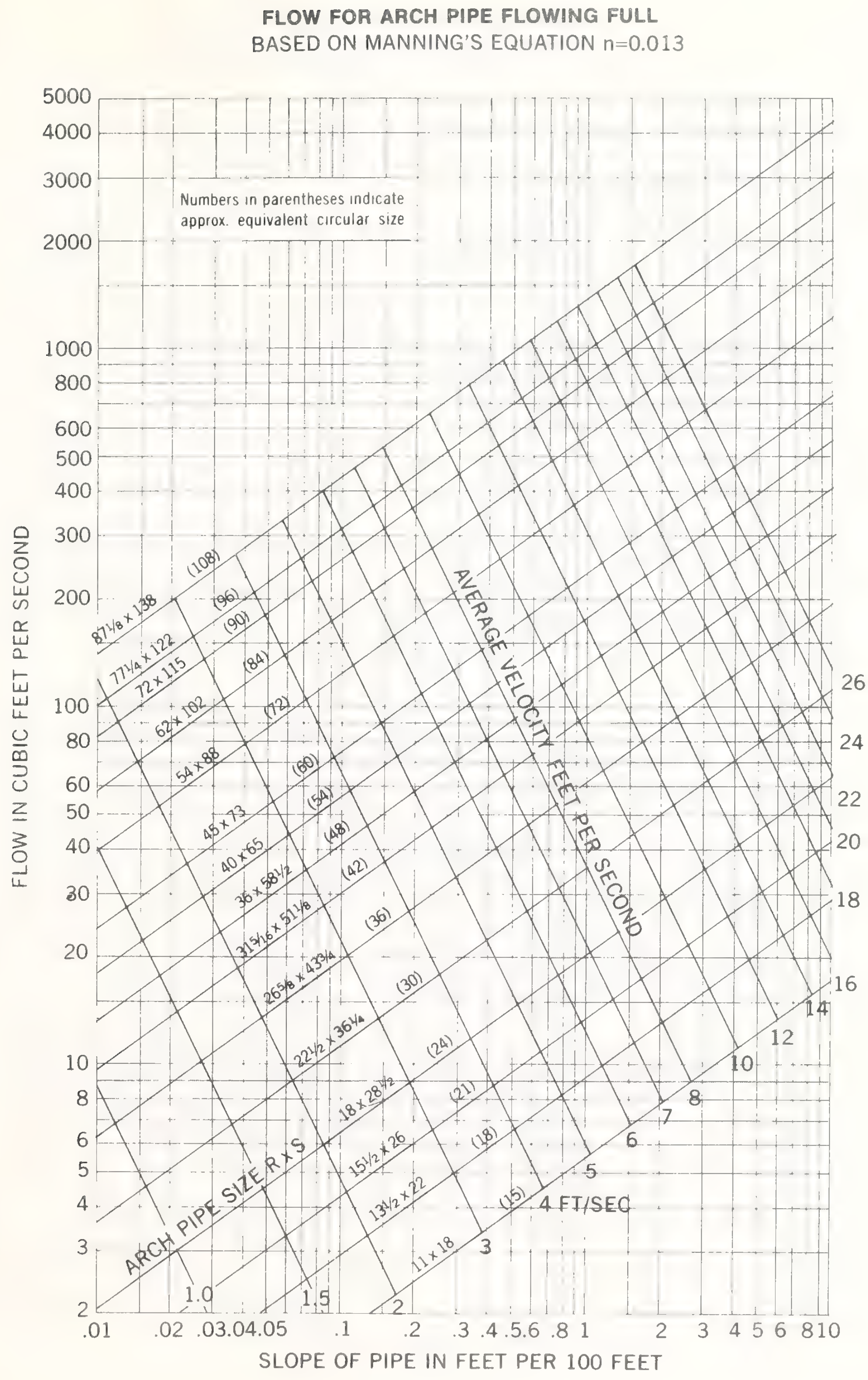


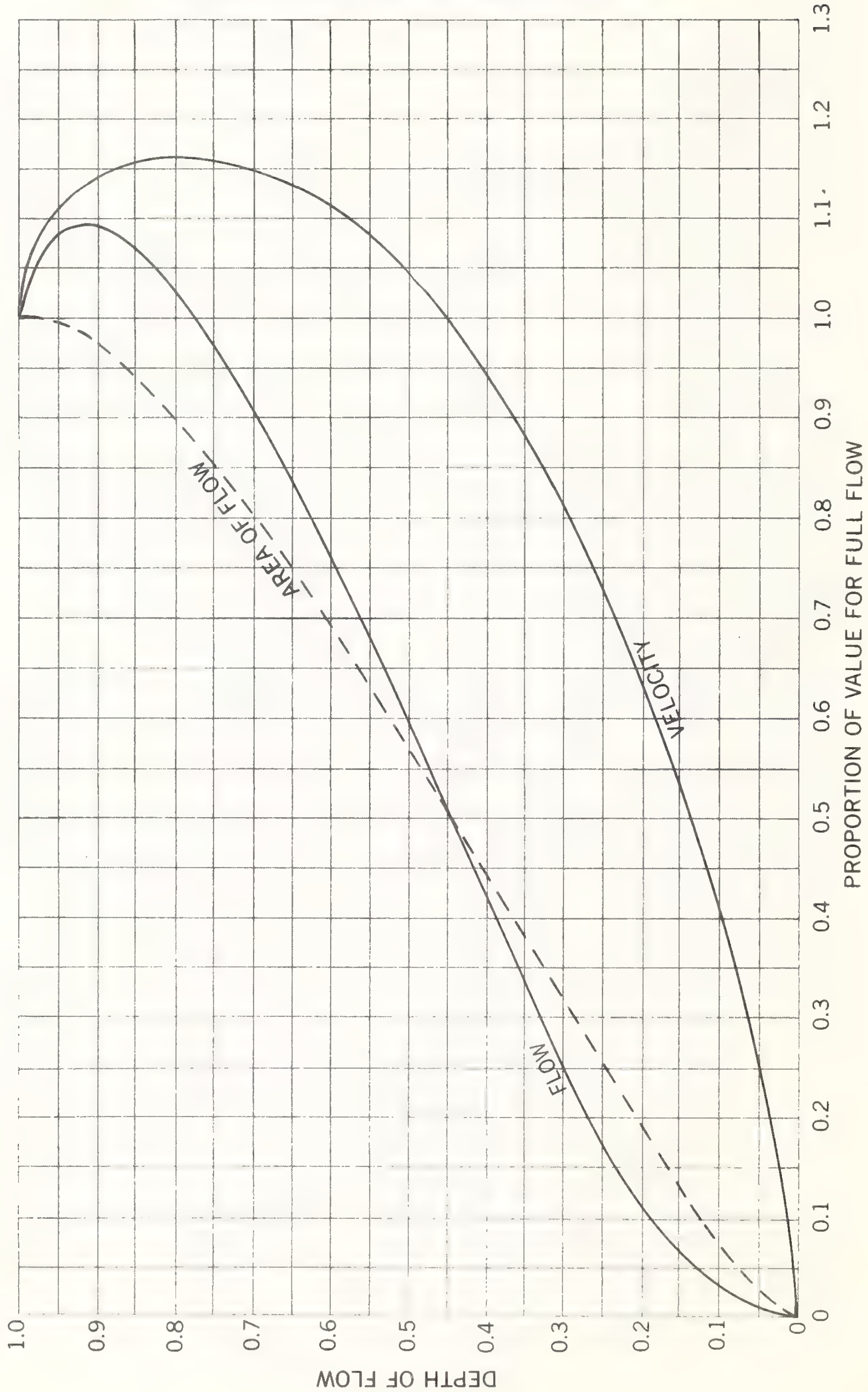
Chart 4.109

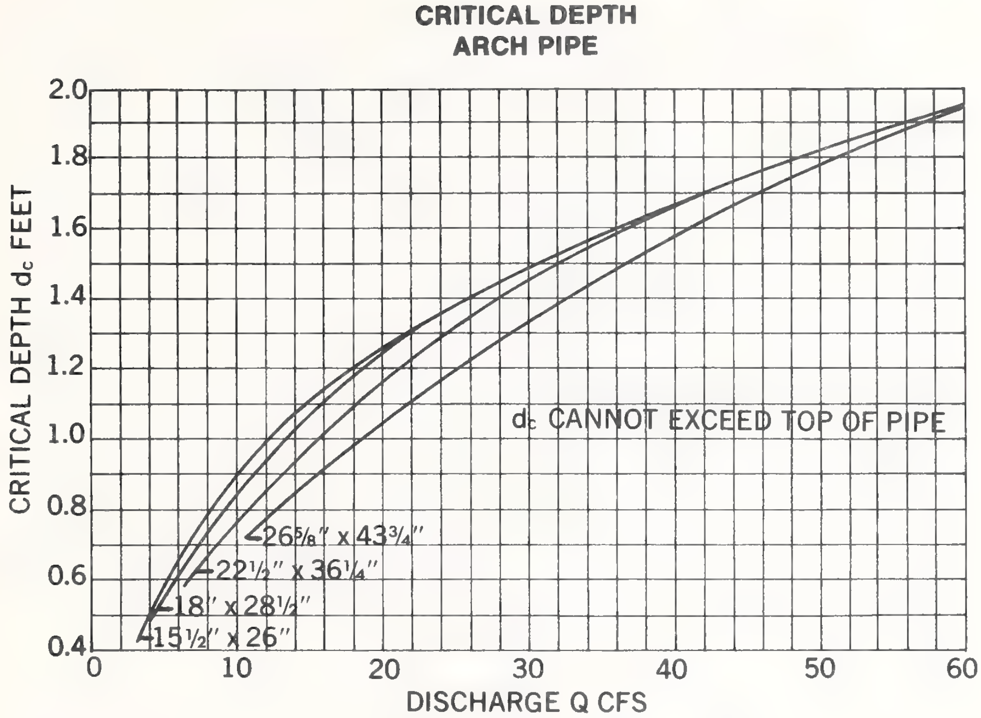


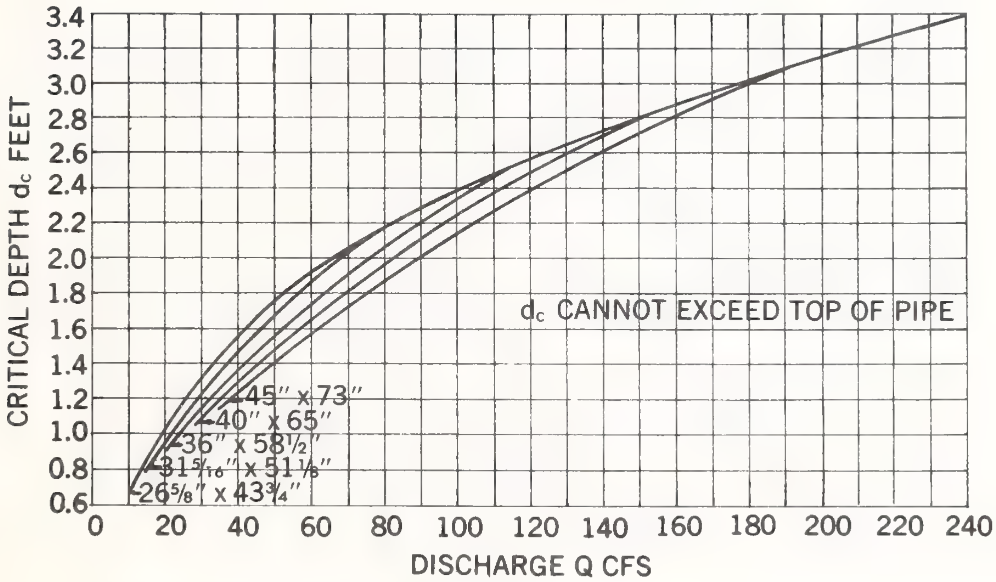
FLOW FOR ARCH PIPE FLOWING FULL
BASED ON MANNING'S EQUATION n=0.013
Numbers in parentheses indicate approx. equivalent circular size
FLOW IN CUBIC FEET PER SECOND
SLOPE OF PIPE IN FEET PER 100 FEET
AVERAGE VELOCITY FEET PER SECOND
ARCH PIPE SIZE R x S
4 FT/SEC

Chart 4.110

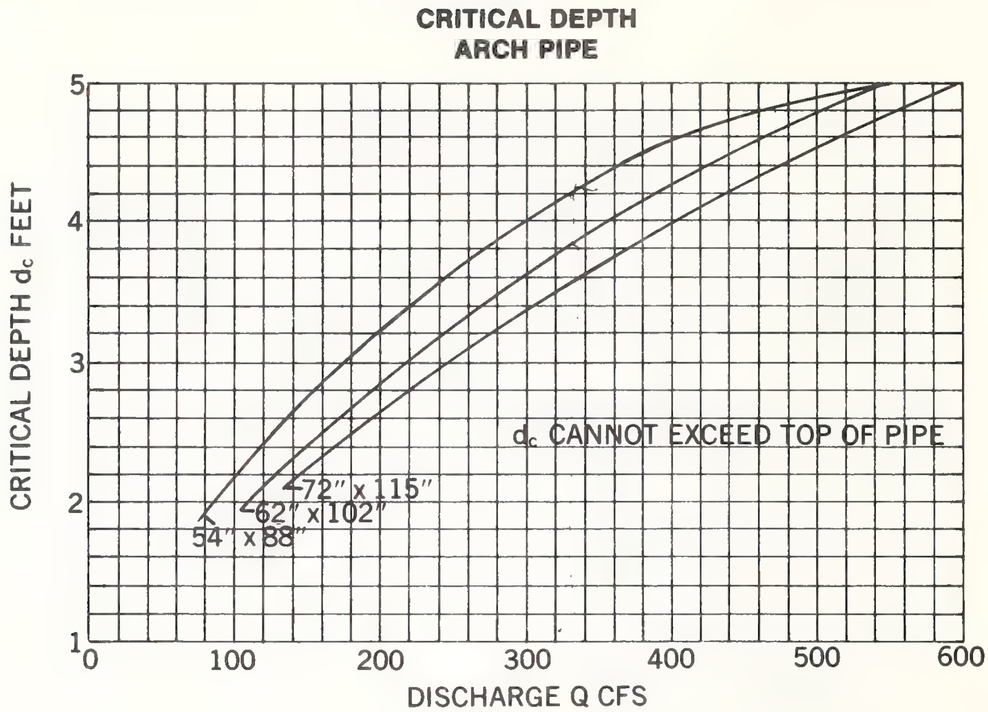
RELATIVE VELOCITY AND FLOW IN ARCH PIPE FOR ANY DEPTH OF FLOW

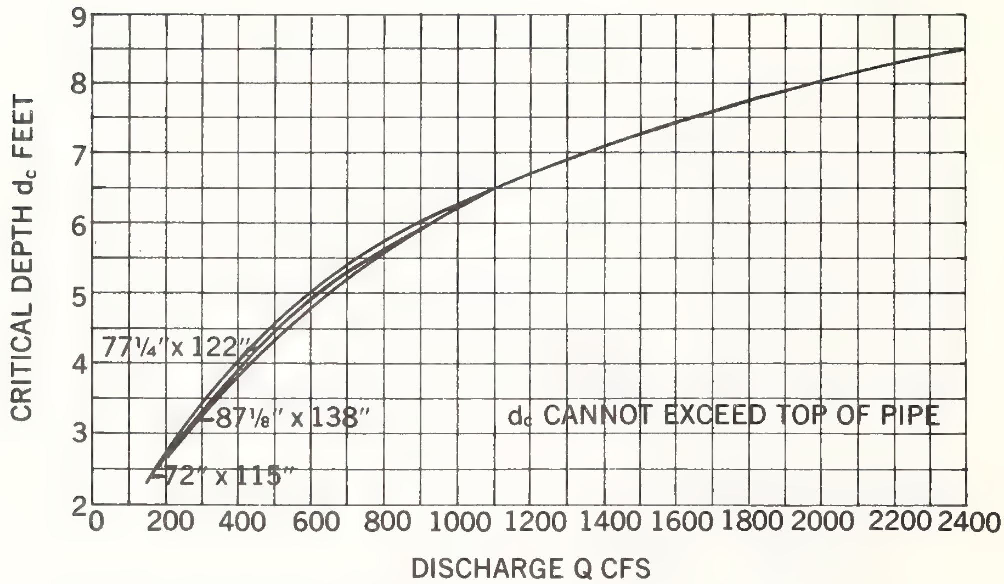






BUREAU OF PUBLIC ROADS, JAN. 1964





BUREAU OF PUBLIC ROADS, JAN. 1964

4.66 OVAL CONCRETE PIPE

Description of Charts - Charts 4.113 - 4.121 are designed for use in the solution of the Manning equation for flow in oval concrete-pipe channels which have sufficient length, on constant slope, to establish uniform flow at normal depth without backwater or pressure head. It is important to recognize that they are not suitable for use in connection with most types of culvert flow, since culvert flow is seldom uniform.

The group consists of Chart 4.113 showing friction slope discharge, and velocity for full flow; Charts 4.114 and 4.115 showing ratios for computing part-full flow; and charts 4.116 - 4.121 used for computing critical flow.

Oval pipe can be laid with the long axis of its cross section either horizontal or vertical. Since the position of the long axis has no effect on flow when the pipe flows full, Chart 4.113 can be used in either case. The position of the long axis does make a difference with part-full flow, however; thus separate charts are necessary for the part-full ratios, and Chart 4.114 is provided for horizontal long axis and Chart 4.115 for vertical long axis. Separate charts are similarly necessary for critical flow, and Charts 4.116 - 4.118 are provided for horizontal long axis and Charts 4.119 - 4.121 for vertical long axis.

It should be noted that a considerable range of pipe sizes are listed at the bottom of Chart 4.113 and all of these sizes are shown on the pertinent charts except Nos. 4.116 and 4.119. On these latter charts, interpolations can be made, when necessary; reading, for a particular size, between the curves for the next larger and next smaller sizes. It should also be noted that dimensions are shown appropriately on Charts 4.116 - 4.121 according to whether the long axis is horizontal or vertical; for example, the pipe shown as 23 by 14 inch on Chart 4.116 is shown as 14 by 23 inch on the corresponding Chart 4.119.

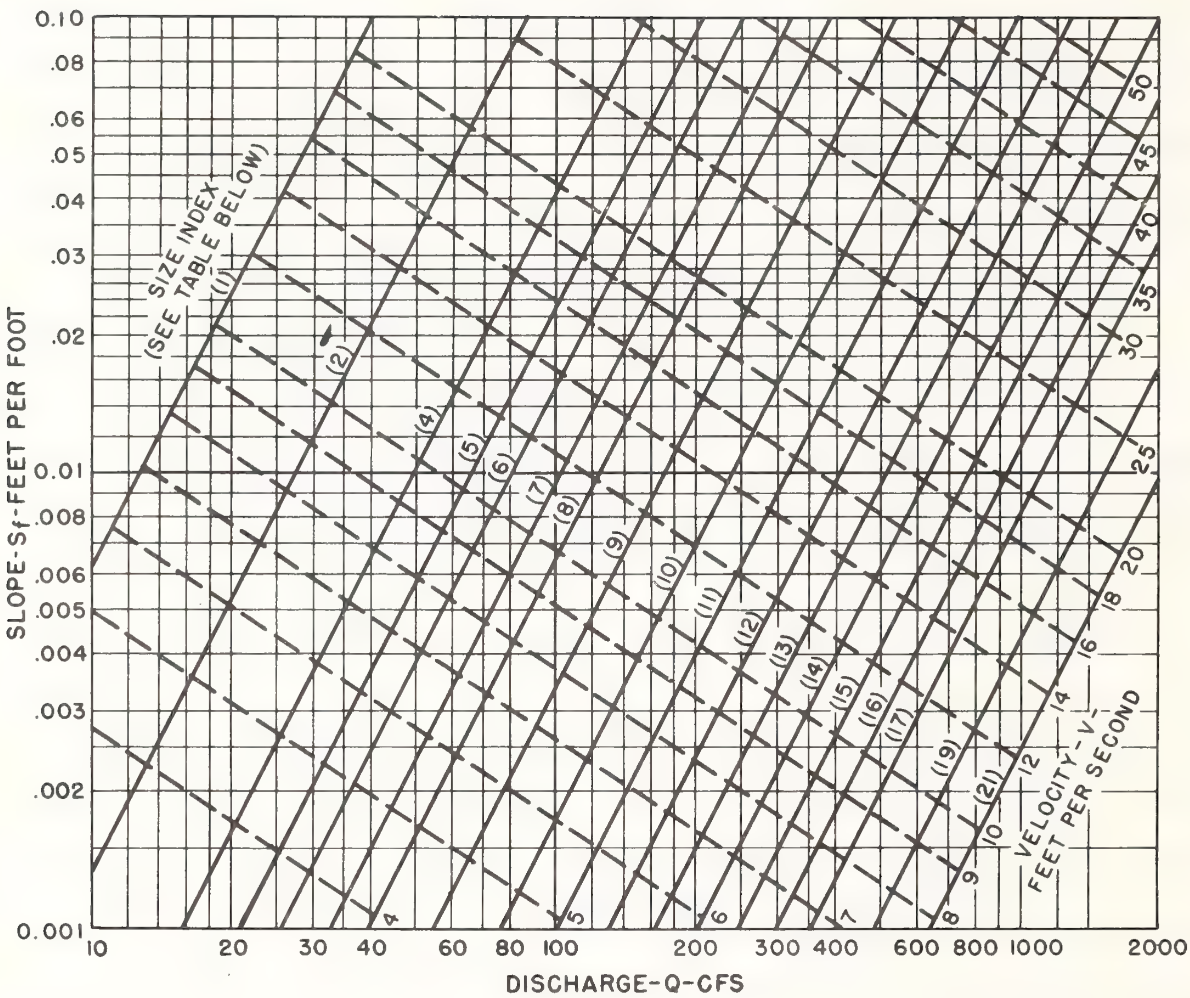
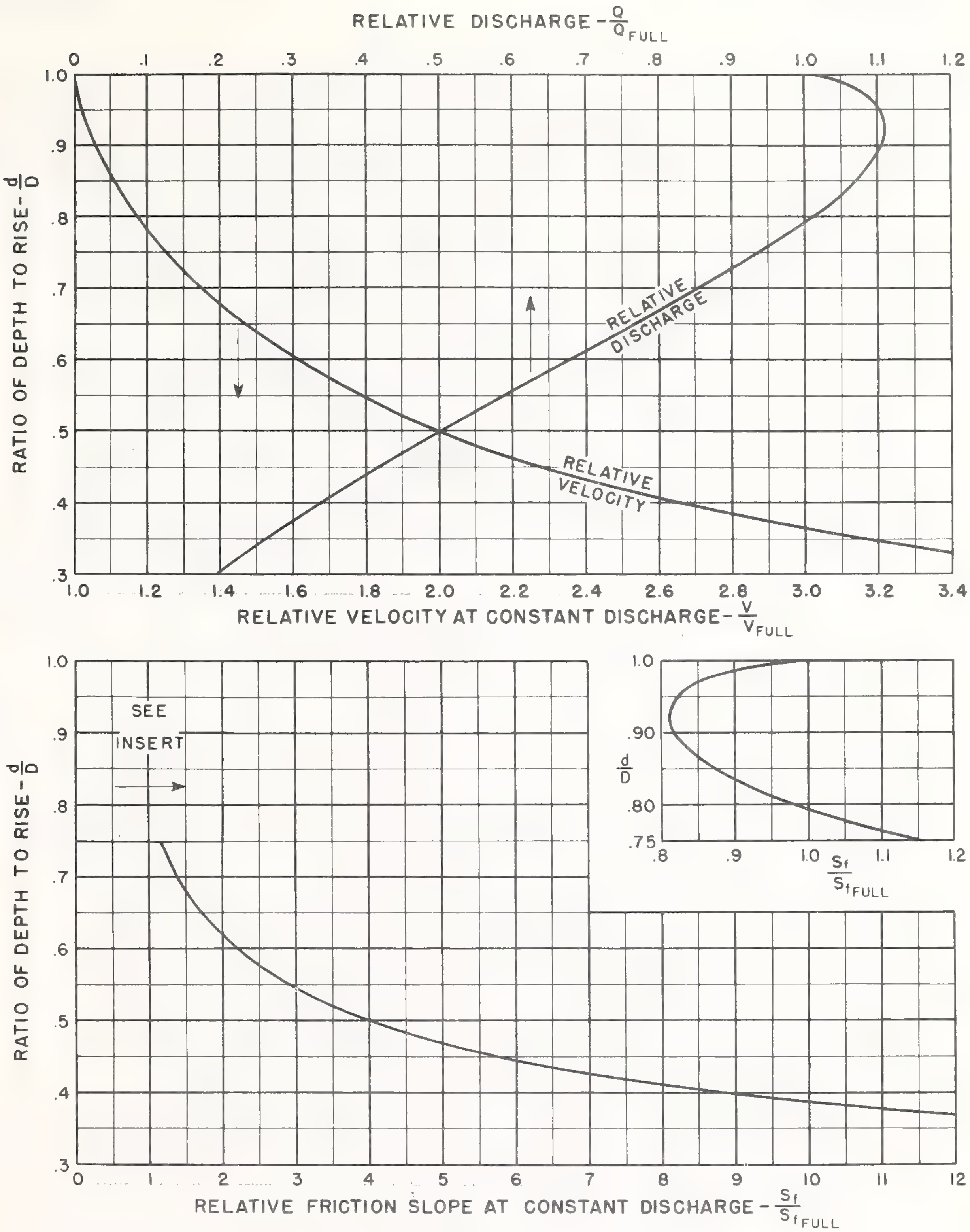


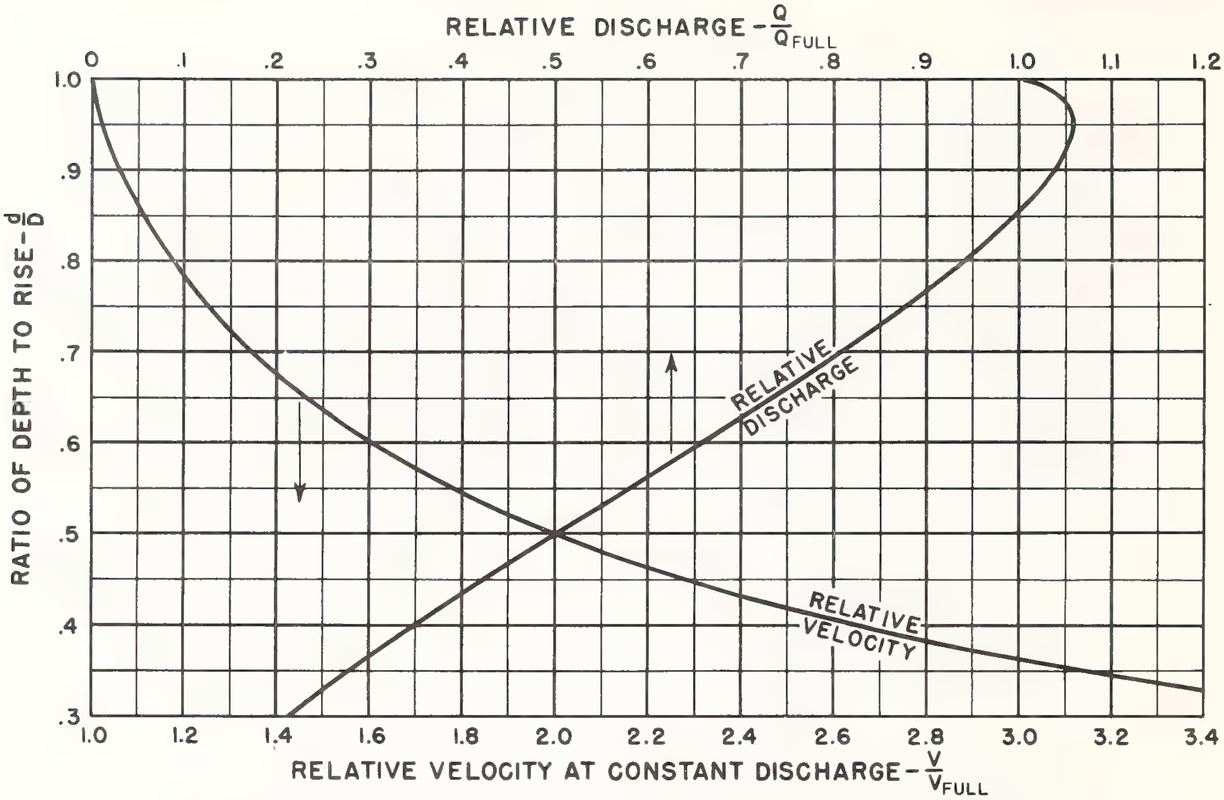
TABLE OF SIZES

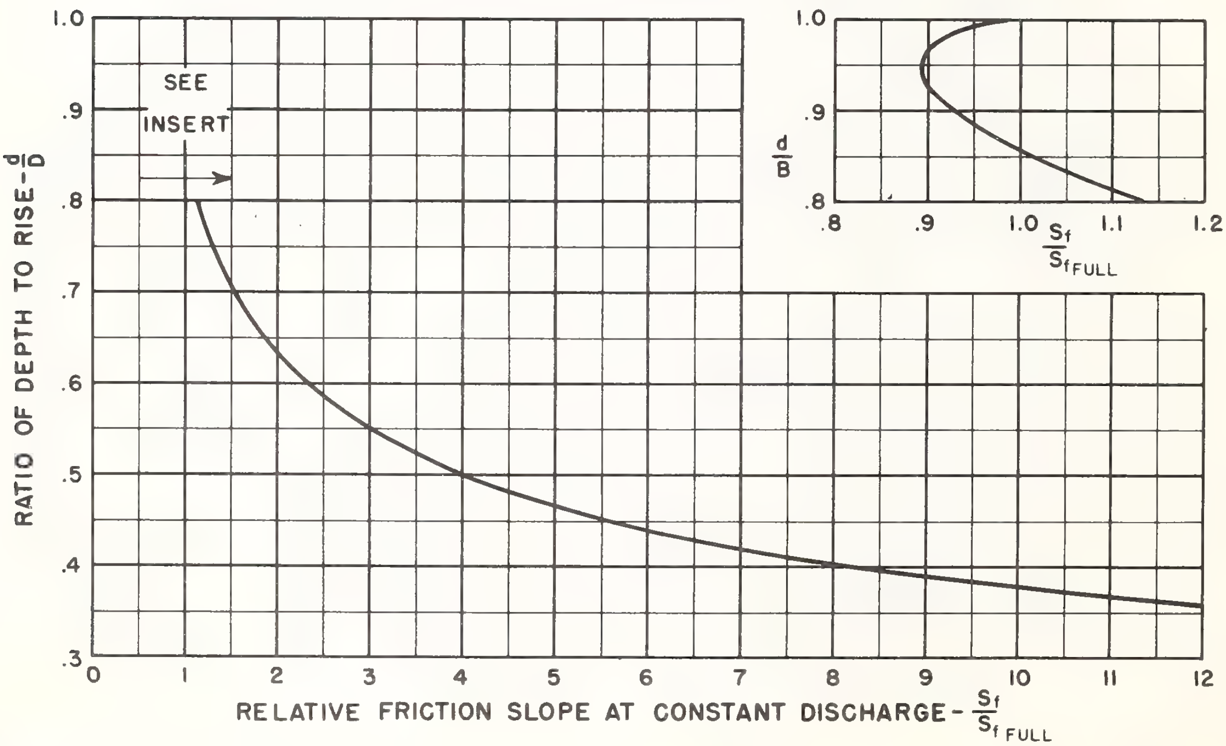
(1)	23" x 14"	(8)	53" x 34"	(14)	98" x 63"
(2)	30" x 19"	(9)	60" x 38"	(15)	106" x 68"
(4)	38" x 24"	(10)	68" x 43"	(16)	113" x 72"
(5)	42" x 27"	(11)	76" x 48"	(17)	121" x 77"
(6)	45" x 29"	(12)	83" x 53"	(19)	136" x 87"
(7)	49" x 32"	(13)	91" x 58"	(21)	151" x 97"

OVAL CONCRETE PIPE
FRICTION SLOPE FLOWING FULL
n = 0.011

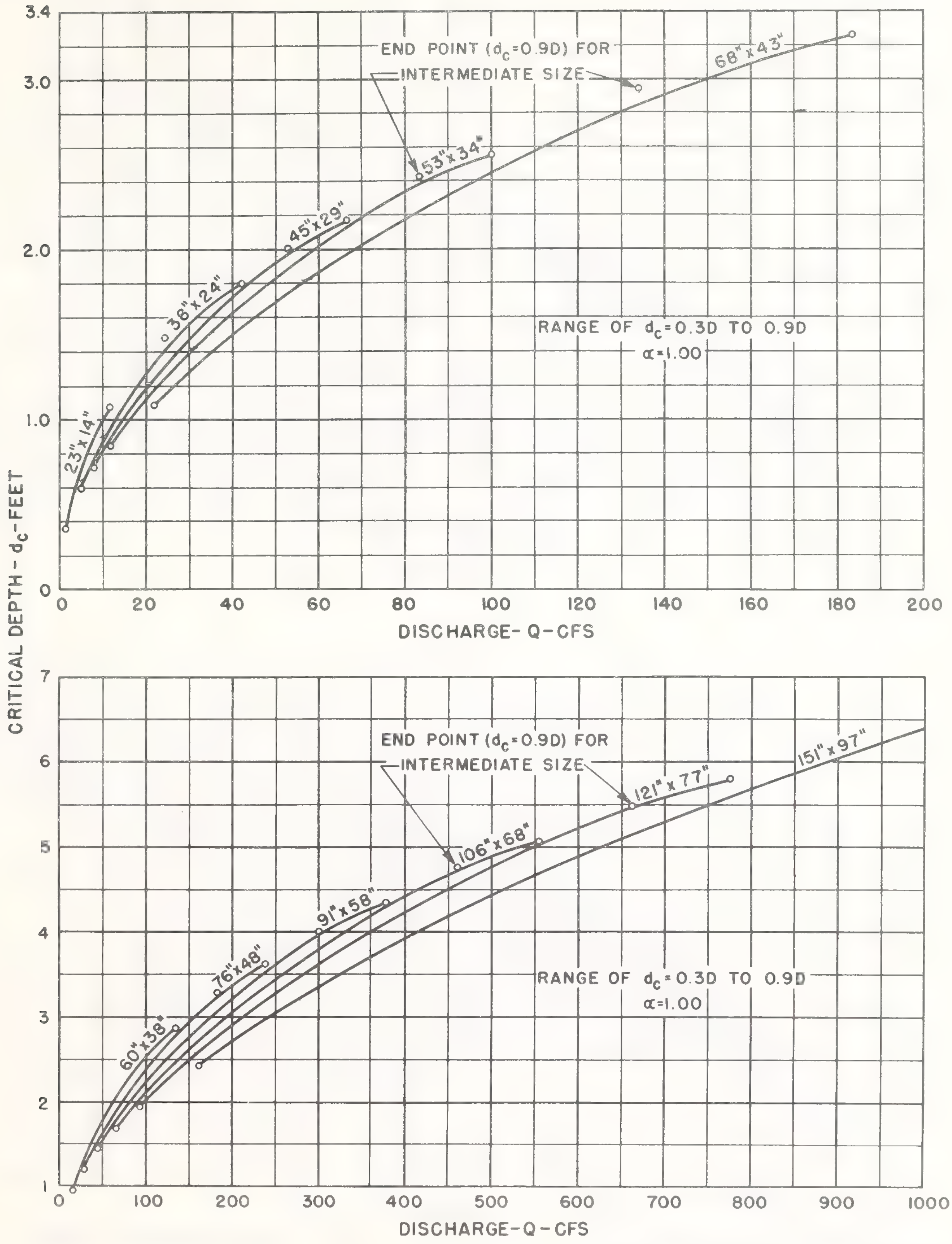


OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL
PART FULL FLOW

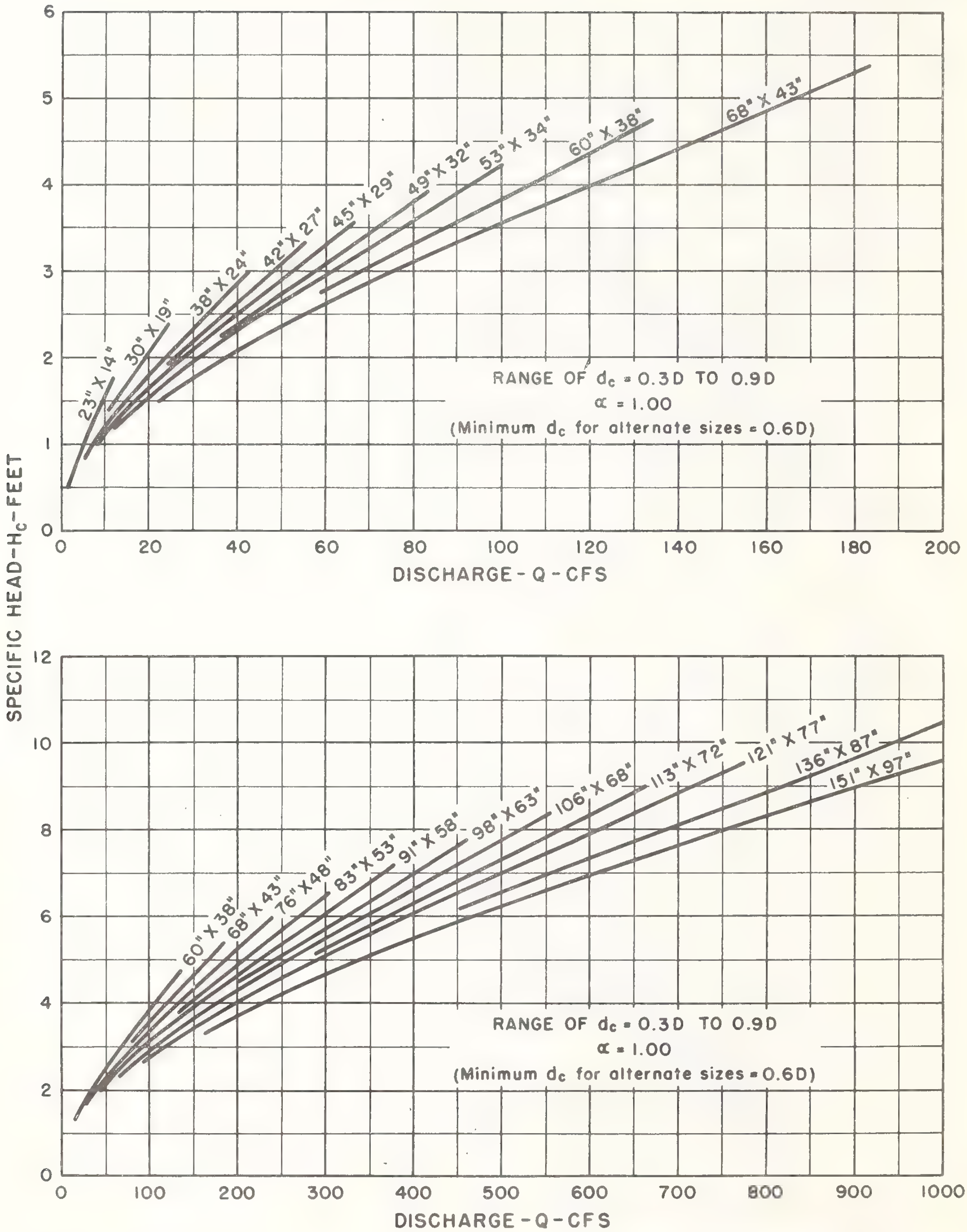




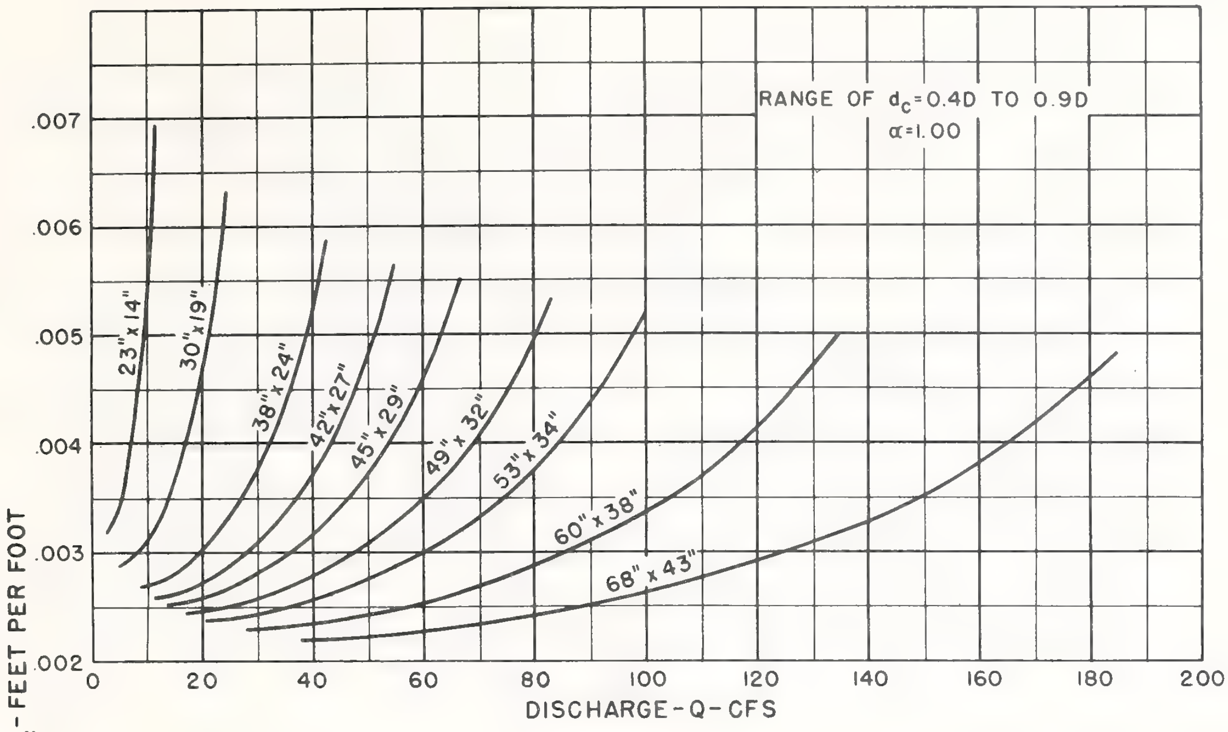
OVAL CONCRETE PIPE
LONG AXIS VERTICAL
PART FULL FLOW

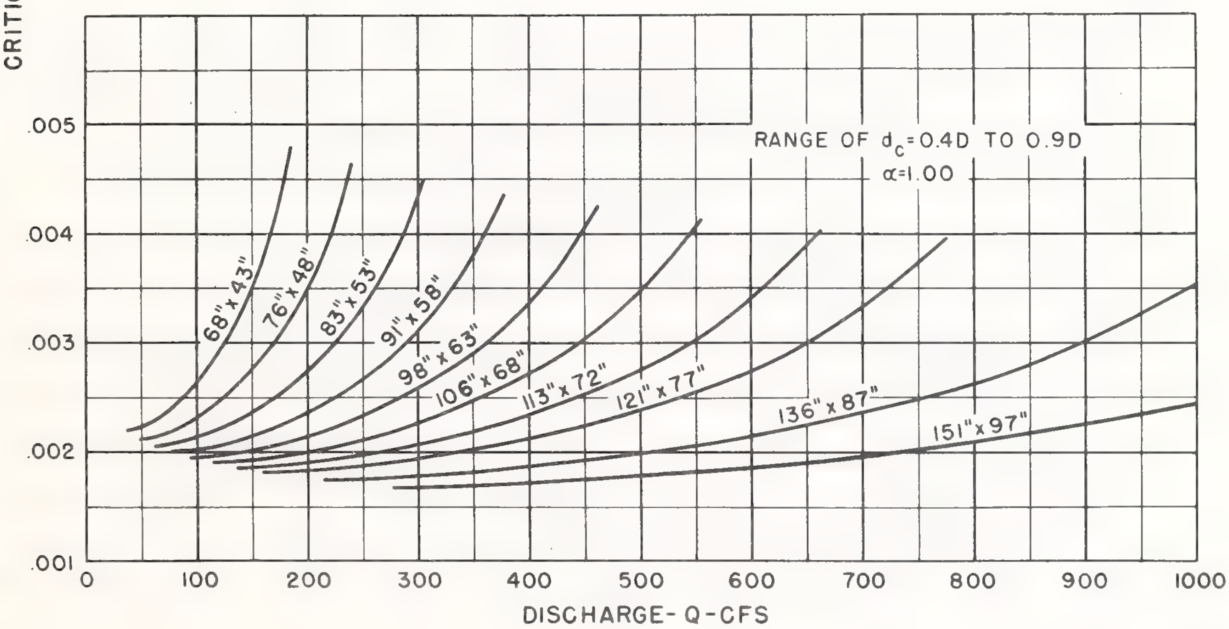


OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL
CRITICAL DEPTH

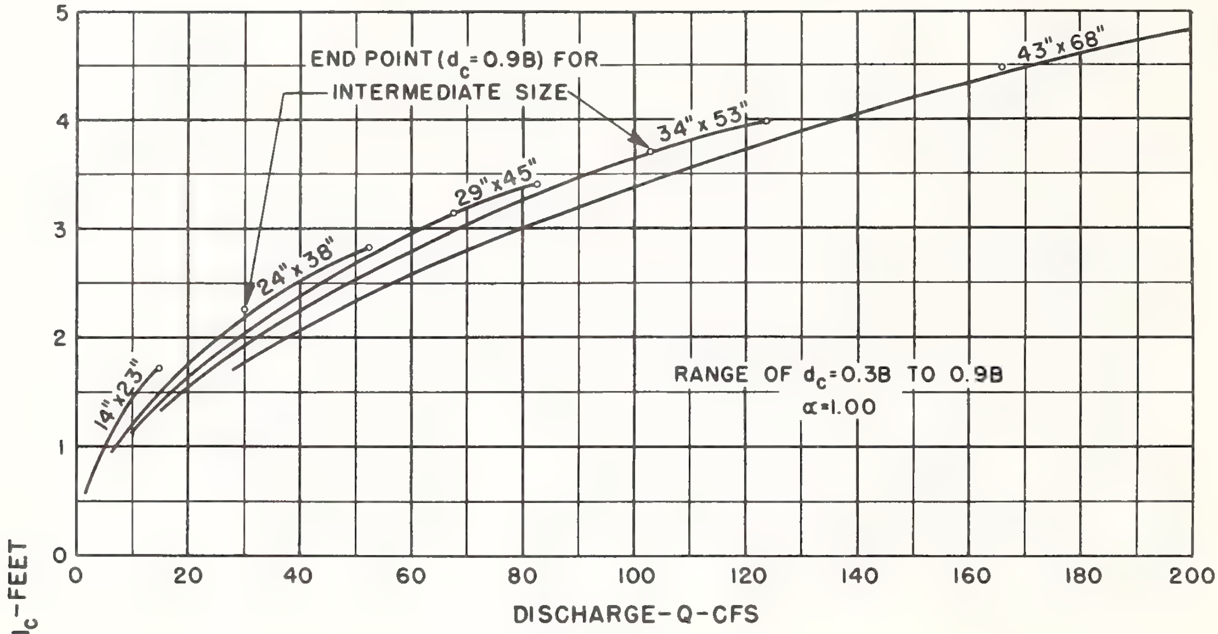


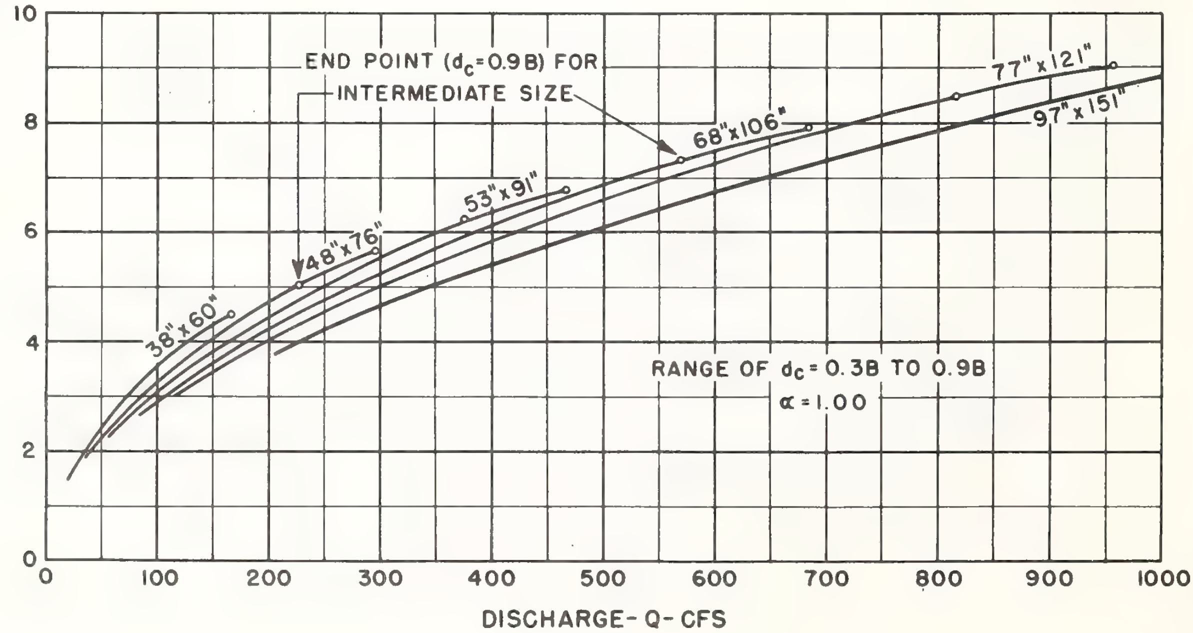
OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL
SPECIFIC HEAD AT CRITICAL DEPTH



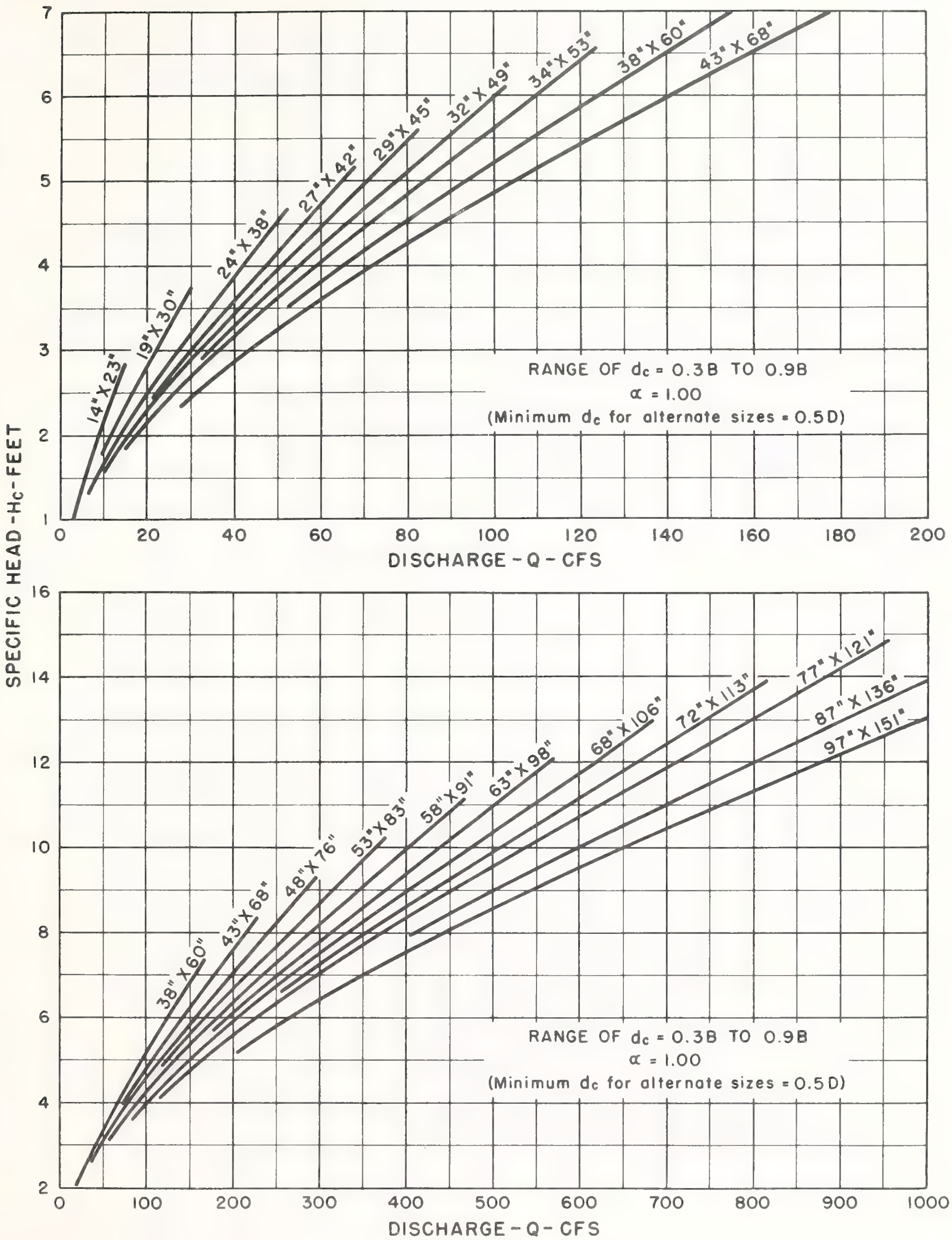


OVAL CONCRETE PIPE
LONG AXIS HORIZONTAL
CRITICAL SLOPE
n = 0.011

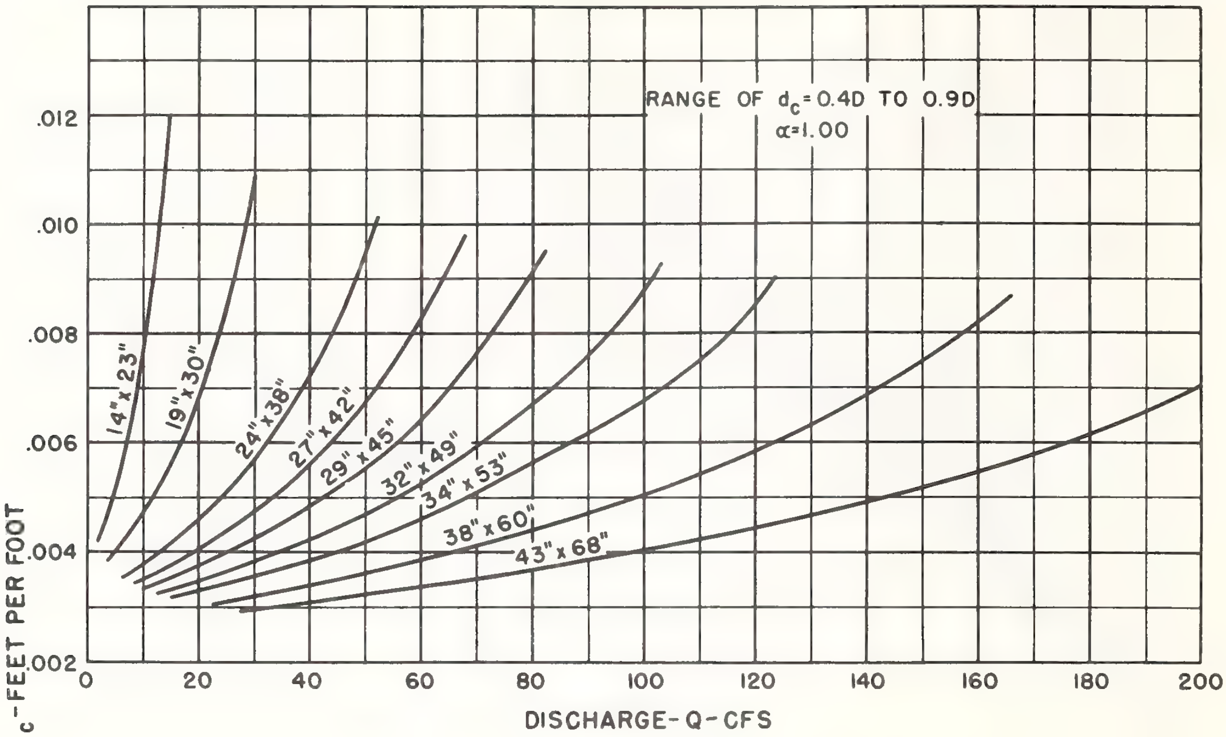


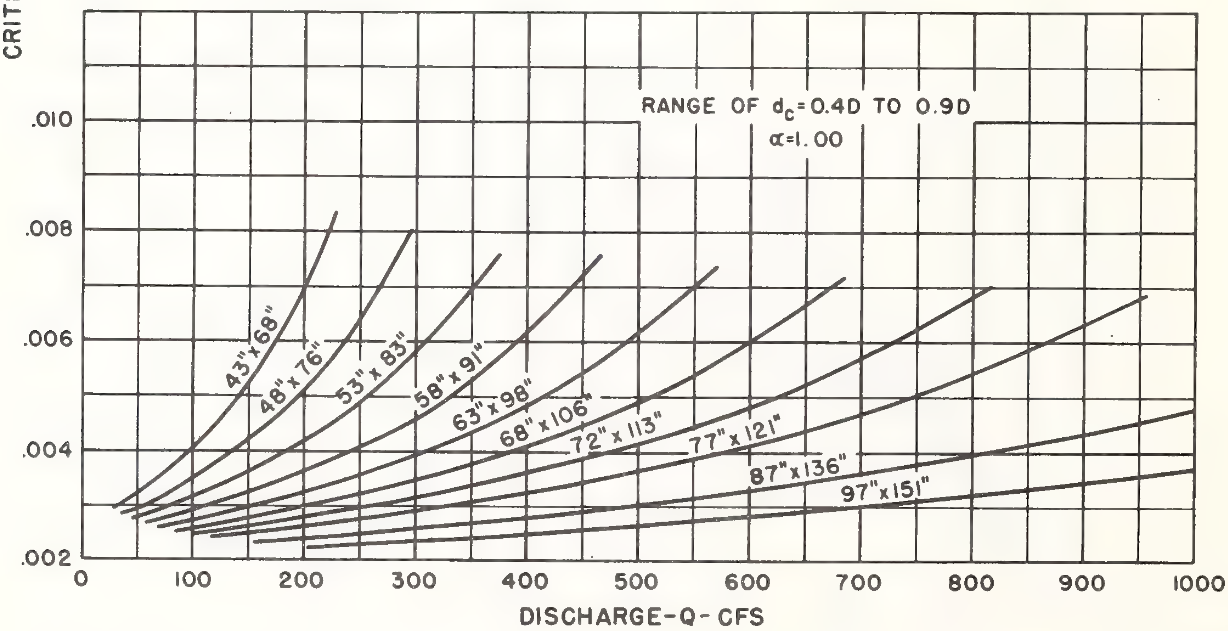


OVAL CONCRETE PIPE
LONG AXIS VERTICAL
CRITICAL DEPTH



OVAL CONCRETE PIPE
LONG AXIS VERTICAL
SPECIFIC HEAD AT CRITICAL DEPTH





OVAL CONCRETE PIPE
LONG AXIS VERTICAL
CRITICAL SLOPE
n = 0.011

4.67 COMPUTER PROGRAMS FOR SOLUTION OF OPEN CHANNEL FLOW PROBLEMS

The following programs will aid in the hydraulic analysis and solution of open channel flow problems.

Name: A   Language: Basic   Input Format: Run

Purpose: Computes and prints the stage-discharge relationship for irregular channels.

Required Input Data: No. of cross section points used to describe the channel, the horizontal distance, elevation and Manning's N for each point, low stage, high stage, channel slope, and design discharge.

Abstract: The program computes the stage-discharge information from the low stage to the high stage at .5 foot increments using Mannings equation. It also computes the normal stage for the designs discharge. The program prints the channel area and velocity as well as the stage and discharges.

Limitations: The program is valid for only uniform open channel flow. A maximum of 25 points may be used to describe the cross section. High stage minus low stage must be 12.5 feet or less.

Example Run

```
        edit a basic
EDIT    run

           STAGE  -  DISCHARGE   RELATIONSHIP

NUMBER OF COORDINATES
  ?   10
DATA
  ?   0,40,.045
  ?   5,38.5,.045
  ?   8,35.1,.04
  ?   13,32.0,.035
  ?   20,31.7,.035
  ?   32,31.9,.035
  ?   40,33.5,.04
  ?   45,35.6,.04
  ?   51,39,.045
  ?   60,40,.045

DESIGN FLOW, CHANNEL SLOPE
  ?   400,.0033

LOW,HIGH STAGE ELEV
  ?   32,39

STAGE ELEV           DISCHARGE            AREA          AVE VEL

  32.00                  2.7              3.46            0.78
  32.50                 25.5             14.03            1.82
  33.00                 67.0             26.26            2.55
  33.50                126.2             40.13            3.14
  34.00                206.1             55.34            3.72
  34.50                303.7             71.55            4.24
  35.00                418.6             88.75            4.72
  35.50                552.5            106.90            5.17
  36.00                705.5            125.81            5.61
  36.50                875.9            145.39            6.02
  37.00               1063.2            165.63            6.42
  37.50               1267.3            186.53            6.79
  38.00               1488.1            208.10            7.15
  38.50               1725.6            230.33            7.49
  39.00               1980.9            253.52            7.81

  34.92                400.0             86.04            4.65

MORE RUNS--YES,1--NO,0
  ?   0
EDIT
```

Name: R22 Language: Basic Input Format: 500 Data, Run

Purpose: Computes the velocity, area, and depth of flow for various flows for triangular, rectangular, and trepezoidal ditches.

Required Input Data: Values of flow are read in with Data Statement No. 500. The bottom slope, half bottom width, left and right ditch slopes, slope of ditch and Manning's *N* are input. The last data entry in Statement 500 must be zero (0) to end the run.

Abstract: The program uses the Manning equation to compute the velocity and depth of flow for all flow values read in on the 500 Data statement.

Limitations: The program is valid only for uniform open-channel flow.

Example Run

```
     edit r22 basic
EDIT  500 data 10,15,20,25,35,50,75,100,200,0
EDIT  run
INPUT BOTTOM SLOPE,HALF BOTTOM WIDTH,LT & RT DITCH SLOPE,DITCH SLOPE,
  ?   0,4,2,2,.0033,.032
DISCHARGE   VELOCITY     AREA    DEPTH   HYD RADIUS    TOP WIDTH
   10.0        1.7        5.7     0.62       0.53         10.5
   15.0        2.0        7.5     0.78       0.65         11.1
   20.0        2.2        9.1     0.92       0.75         11.7
   25.0        2.4       10.6     1.05       0.83         12.2
   35.0        2.6       13.3     1.27       0.98         13.1
   50.0        2.9       17.1     1.54       1.15         14.2
   75.0        3.3       22.8     1.92       1.37         15.7
  100.0        3.6       28.0     2.24       1.55         17.0
  200.0        4.3       46.2     3.20       2.07         20.8
EDIT  end
READY
```

Name: HEC-2 Language: Fortran Input Format: Code Sheets

Purpose: Computes water surface profiles for river channels

Required Input Data: The required input data and coding procedure are much to complex to detail here. The Users Manual must be used when coding a project.

Abstract: The program entitled "HEC-2, Water Surface Profiles" was written by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The program computes and plots the water surface profile for channels of any cross section for either subcritical or supercritical flow conditions. The program will handle both uniform and non-uniform flow. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments, and dams may be considered in the computations. The Users Manual must be consulted when using this program because of its complexity.

Limitations: See Users Manual

Example Run: See Users Manual

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SECTION 4.7
URBAN STORM DRAINAGE

4.7 URBAN STORM DRAINAGE

Introduction

This section has been prepared to present an accepted approach to the design of storm sewers for urban highway projects in Montana. Much of the information utilized in this section is the result of empirical formulas and experimentation. Because of this, a rigid mathematical application of the principles outlined would result in refinements of design beyond the limitations of basic data. As in all cases of good design, the results obtained must be tempered with good judgement. All of the contingencies of storm sewer design cannot be covered in this manual due to the number of such contingencies.

A storm sewer system is a drainage structure designed to prevent the accumulation and retention of water on highways and other surfaces and to prevent the discharge of accummulated waters onto abutting landowners. To be a properly designed functional drainage system, the facility must incorporate the following desirable characteristics:

Surface and subsurface drainage must be provided to assure the stable soil conditions necessary for adequate soil bearing capacity.

Surface runoff must be removed with little damage to highway facilities and insignificant interruption of normal traffic.

Storms of greater intensity than the design storm must be removed with the minimum damage and the least interruption to normal traffic that is practical.

Maintenance and operation difficulties must be minimized.

Future expansion of facilities with a minimum of expense or interruption must be possible.

Storm water must be discharged with a minimum of damage to the receiving stream.

The facility provided must represent the best compromise between economy and serviceability.

The storm sewer system necessary to provide the above requirements consists of the following minimum components:

Provisions for interception and diversion of storm runoff from areas continguous to the highway.

A system of curbs and gutters to convey surface runoff to points of inlet to the underground storm sewer.

Inlets to admit water from the gutters to the storm sewer.

A storm sewer to collect the flow from drains and branches and to convey the water to a point of disposal.

Outlet structures suitable for disposal of discharge with minimum erosion and damage to the receiving stream.

4.71 DESIGN APPROACH

When the basic data outlined in Section 2.6, "Urban Storm Drainage Surveys," has been accummulated, the design may proceed. The following general outline presents a design approach:

1. Determine the general drainage patterns.
2. Get city involved in project design.
3. Determine the design storm.
4. Locate inlets and establish drainage sections.
5. Determine inlet time and runoff for each drainage section.
6. Establish preliminary storm sewer layout.
7. Establish a profile with control elevations.
8. Determine required capacity, slope of pipe, diameter, and velocity of each pipe section.
9. Design outfall.

The order of many of the items in this outline is flexible and some overlapping of content exists. The following discussion more thoroughly covers items in this outline and will acquaint the designer with some of the considerations necessary for storm sewer design.

Determine the General Drainage Patterns - The general area that will be served by the storm sewer should be determined and the street slopes and drainage patterns identified. Potential locations for discharging the storm water should also be located. This general information gives the designer a better idea of how and where to begin the design.

Get the City Involved in Project Design - The city or local government should be contacted early in the design to see if they wish to participate in the project by having the storm sewer pick up city drainage. If the city participates and city drainage is provided for, an agreement between the State and City for financing of the project must be made.

Determine the Design Storm - One of the most difficult problems to resolve in beginning a storm sewer design is the storm frequency to use. Storm frequency may be defined as the number of years that will probably pass before a storm of given intensity and duration will occur. It is generally considered impractical to design all storm sewer systems for the maximum intensity expected every 20 or 25 years. The inconvenience of overloading once or twice in this period is usually less objectionable when compared against the economics of providing a system capable of handling the maximum intensity to be expected.

The frequency of design runoff to be utilized for a storm sewer system should range from once in two years to once in ten years. Table 4.35 sets guidelines for storm frequency.

Table 4.35

Storm Sewer Design Storm Frequency

Land Use	Design Storm Return Period
Residential	2 Years
High Value General Commercial Area	5 Years
Public Buildings Area	5 Years
High Value Downtown Business Area	5-10 Years

With the storm frequency determined, the storm intensity - duration data can be determined using the figures and procedures of Section 3.5, Rational Method of Runoff Prediction.

Locate Inlets and Establish Drainage Sections - The locations where inlets will be required and the area that each inlet will drain should be determined.

Storm inlets admit water from gutters into the sewer system. Inlets must be placed so that the storm water is picked up before it can inconvenience traffic or pedestrians and before it can cause flooding. In general inlets should be placed at the following locations:

1. Prior to all pedestrian crossings.
2. At all traffic intersections.
3. At all low points in the gutter grade.
4. Where significant flows from off the right-of-way (side streets, parking lots, etc.) are expected.
5. On horizontal curves where a change from normal crown to superelevation may cause water to cross the highway.
6. Where lay-down curb may allow the storm water to escape and cause flooding
7. Where gutter flows become so great that more than half of the driving lane is flooded during a 10-year storm.
8. Where the highway fill causes ponding that cannot be drained in any other way.

There are two basic types of inlets that can be used, the drop inlet with grate and corrugated steel slotted drain. Details of both types of inlets are

shown in the Standard Drawings.

The drop inlet and grate can be used in two ways, with the bars in the grate parallel to the curb and with the bars transverse to the curb. When the inlet is used with bars parallel to the curb, it has good hydraulic capacity but may present a hazard to the bicycling public. Therefore, this inlet should be used with the bars parallel to the curb only when bicycles are prohibited from using the highway or when separate bicycle paths are provided. When the drop inlet is used with bars transverse to the curb, storm flow tends to jump over the grate and very little is intercepted. However, when the inlet is placed in the bottom of sag vertical curves, the orientation of the bars is of little importance and its hydraulic capacity is good. Therefore, the drop inlet with the bars transverse to the curb should be used only in sags. The capacity of the grate used in this manner can be predicted from the weir formula:

$$Q = C_W \times L \times h^{3/2}$$

where Q = inlet capacity in cfs

C_W = weir coefficient = 3.0

L = perimeter of grate over which water flows 7.5 ft.

h = depth of water over grate in feet

The capacity of the drop inlet and grate is significantly increased if the grate is depressed. Therefore, it is recommended that the grate be depressed whenever it is not to be placed in a driving lane.

The corrugated steel slotted drain inlet will safely pass the bicycle and therefore can be used where bicycle traffic is expected. Flow into the slotted drain is a combination of weir flow and orifice flow. Computer program, "SDC Basic" can be used to solve the weir and orifice flow equations and determine the length of slotted drain necessary to intercept known flow on known slope.

With the location of the inlets established, the drainage section for each inlet can be determined. A drainage section is an increment area of the drainage pattern which contributes runoff to one inlet. Drainage sections, once established, should be drawn on the map showing drainage patterns and designated in a manner which identifies it with the appropriate inlet.

Determine Inlet Time and Runoff For Each Drainage Section - Storm runoff is controlled by the weather, the condition of the soil, the land use and type of surface, the drainage area, the general slope of the ground, the duration of the storm and the existance of natural or man-made obstructions. In general, design storms in Montana have been found to occur in July or August. In certain areas, however, a frozen surface condition could require consideration. Neglecting the possibility of a design storm falling on snow or frozen soil would result in questionable design only if such a condition occurred in a period less than the design frequency. Because such a critical condition is highly improbable, pracitical design considerations require the use of a normal unfrozen soil condition. Table 3.4 shows runoff coefficients for various surface materials and land uses. As the runoff coefficient will probably increase with storms of longer duration, due to partial saturation of the soil, the tabular values should be considered only as a guide. Extensive residential or open urban areas require careful consideration in determination of runoff factors. Possible future changes in land use should be considered when determining runoff coefficients.

With drainage sections and slopes known, the inlet time is determined. "Inlet time" is the period required for a particle of water falling at the most remote point of the drainage section to reach the section inlet by following the normal drainage pattern. Section 3, Hydrology, presents several methods for the determination of flow time from the most remote point in the drainage section to the inlet. A minimum of five minutes should be used to allow for initial rainfall absorption.

With the inlet time known and the intensity - duration data previously determined, the design intensity can be determined for each section. Using the Rational Formula as outlined in Section 3, Hydrology, and the drainage area, runoff coefficient, and design intensity the runoff for each inlet can be computed. The inlet and the connection from the inlet to the manhole should be designed for this flow.

Established Preliminary Storm Sewer Layout - The location of the manholes and the pipe can now be shown on the plans. The placement of these components is discussed in detail in the following paragraphs.

To effect future repairs and eliminate manhole covers in driving lanes, it is desirable to locate new storm sewers outside the pavement area. Medians usually offer the most desirable storm sewer location. In the absence of medians, a location beyond curb line on state right-of-way or on easements is preferable. Desirable locations outside the paved area are frequently occupied by other utilities or obstructions and usually cannot be used for storm sewer location. In the absence of a median or suitable unpaved area, storm sewer location must be considered under the highway surface. In this event, the least hazardous area of the pavement, where damage is unlikely to occur and repairs can be most conveniently and inexpensively made, must be chosen. The desirable storm sewer location under pavement is that area abutting the gutter lip, allowing sufficient clearance for making the proper inlet to storm sewer connection.

Reinforced concrete sewer pipe shall normally be used for storm sewers. *All sewer pipe shall be the rubber gasket joint type. This minimum diameter of pipe used shall be 12 inches for inlet to manhole connectors and 18 inches for all sewer lines.

Manholes are required in the system to facilitate inspection and cleaning. They should be provided at junctions of sewer lines, at changes in alignment, at changes in grade, and at changes in pipe diameter. Head losses occur at

changes in alignment and a drop in grade of 0.1 foot is usually allowed in the manhole. The junction of two pipes of different diameter is made by matching top inside elevations in the manhole. This procedure eliminates surcharge in the smaller pipe which would occur if inverts were matched.

If desirable, lines from catch basins may enter the manhole at any convenient point above the storm sewer invert. Manholes should be placed at intervals up to 500 feet unless spacing is controlled by one of the previously discussed considerations. Spacing of from 1,000 to 1,200 feet is permissible in sewers of 48 inch diameter or more because the large sizes allow maintenance crews to enter the line for inspection, cleaning, and repairs. Alignment for these larger diameter sewers may be curved because of the ease of entering the line for maintenance purposes.

<u>Establish Profile with Control Elevations</u> - Before design of the main sewer begins a profile should be established which shows all control elevations. Control elevations include anything that might affect the selection of the sewer profile.

Minimum cover to protect the sewer pipe from excessive loads must be provided along the sewer pipes and on inlet connections. The cover shall never be less than that indicated on the tables of Section 4.14 and pipe should not be allowed to extend into the surfacing courses. A cover of at least two feet is preferable where practical. Pipe class or metal thickness must be selected to fit the minimum cover provided.

The location and elevation of utilities and other obstructions should be established on the profile so conflicts can be minimized. Some utilities such as power lines, telephone lines, and small water lines can be moved to avoid the storm sewer unless they are encased in concrete. Other utilities such as sanitary sewers, other storm sewers and large water lines must be avoided. Railroads often require clearances that should be shown on the profile.

The outlet control elevation is set by the receiving stream. The possible water surface elevations in the receiving stream must be considered when setting this control elevation and when designing the storm sewer outfall.

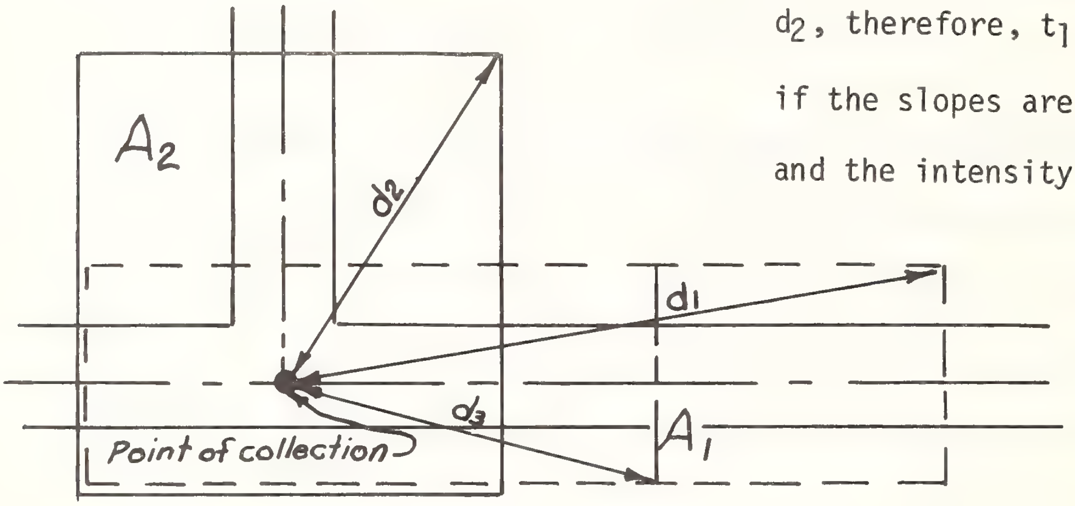
With the proper designation of all control elevations on the profile, these elevations are more easily considered during the sewer design.

Determine Required Capacity, Pipe Diameter, Slope, and Velocity of Each Pipe Section - As the design of each section of the storm sewer depends upon the characteristics of the previous sections, the design must start at the upper most part of the sewer and proceed downstream a section at a time. The Storm Sewer Design Sheet provides an outline for an orderly approach to storm sewer design.

The required capacity of a section is dependent upon its "time of concentration" and contributing drainage area. The time required for the maximum runoff rate to develop in any section of the storm sewer is designated the "time of concentration" and is equal to the time required for a drop of water to run from the most remote point of the water shed to the point for which runoff is being estimated. It is apparent that this period is a function of inlet time, flow time in the sewer, storm intensity and duration. A careful consideration of all factors is necessary for its proper determination. The time of concentration used to determine sewer size and slope for a sewer without branches is the inlet time at the most remote point plus the total flow time in the sewer. The design concentration time for a point below the junction of two or more sewer branches is not necessarily the longest of the two periods. A larger flow could easily result with the smaller concentration time. A long narrow area at the upper end of the branch having the longest concentration time would contribute little additional flow but the longer duration would result in a lower intensity and a probable lower design flow than from the other area. This factor is more easily seen by considering two equal drainage areas, a

square area for the section having the shorter concentration time and a long rectangular area for the section having the longer concentration time. It is readily

$A_1 = A_2$, $d_2 = d_3$, d_1 is greater than d_2, therefore, t_1 is greater than t_2 if the slopes are approximately equal and the intensity for A_2 is higher.



apparent that the higher rainfall intensity on the square area caused by the shorter concentration time results in a larger flow than is contributed by the lower intensity, longer storm on the rectangular area. However, that portion of A_1 up to distance d_3 will also be contributing at the same rate as A_2. The runoff from the square area is past the junction when the maximum runoff from the rectangular area occurs. All conditions must be investigated when determining time of concentration (t) for any multiple storm sewer design.

The intensity of a storm having a duration equal to the time of concentration can be determined from the previously established intensity - duration data and is applied to the appropriate drainage areas. Runoff coefficients previously established are used to determine what portion of this rainfall will reach the storm inlet. The junction of flows from more than one inlet may require a recalculation of quantities, depending upon which time of concentration controls the combined flow.

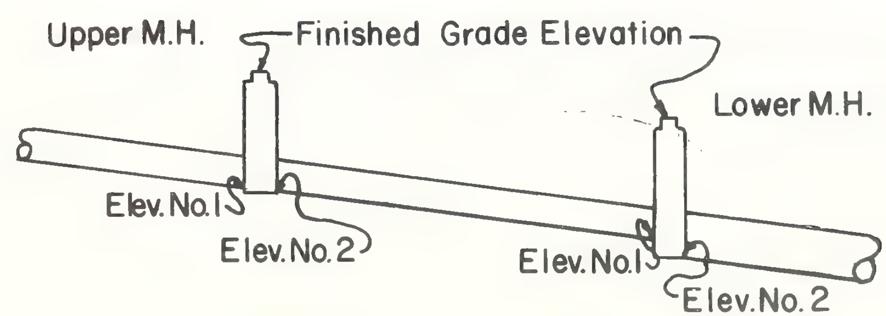
With the flow in the section under consideration known, a pipe diameter and slope may be selected to accommodate this flow. The open channel flow

STORM SEWER DESIGN SHEET

N = ______________ DESIGNER ______________ PROJECT ______________

DESIGN FREQ. ________ YRS. DATE ______________

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
																	INVERT ELEVATIONS			
LINE NO.	LOCATION	STATION	INCR. AREA "A"	"C"	INCR. CxA	TOTAL C x A	TIME OF CONC. MIN.	I IN./HR	Q C.F.S.	PIPE SLOPE FT./FT	DIA. OF PIPE INS.	V FT/SEC	L FT.	TIME OF FLOW MIN.	CAP. OF SEWER C.F.S.	FINISHED GRADE ELEVATION AT MANHOLE	ELEV. NO. 1	ELEV. NO. 2	M.H. DIA.	



nomographs for pipe sections found in Section 4.6 of this manual will aid in the selection of the diameter and slope. Manning "n" of 0.012 should be used for concrete pipe. When the depth of flow in the pipe exceeds 75% of the pipe diameter the next larger size pipe should be used. The pipe diameter and slope should be selected so that the velocity in the pipe when it is flowing full is greater than 2.5 ft/sec and less than 10.0 ft/sec., when possible. The diameter and slope must also be established to fit all control elevations.

Manhole diameters should be specified that will give easy access to the sewer and will give enough room to join all pipe. The manhole should be large enough to maintain one foot of clearance between all incoming pipes as indicated in Figure 4.49 with due consideration of vertical alignment and with a minimum diameter of 48 inches.

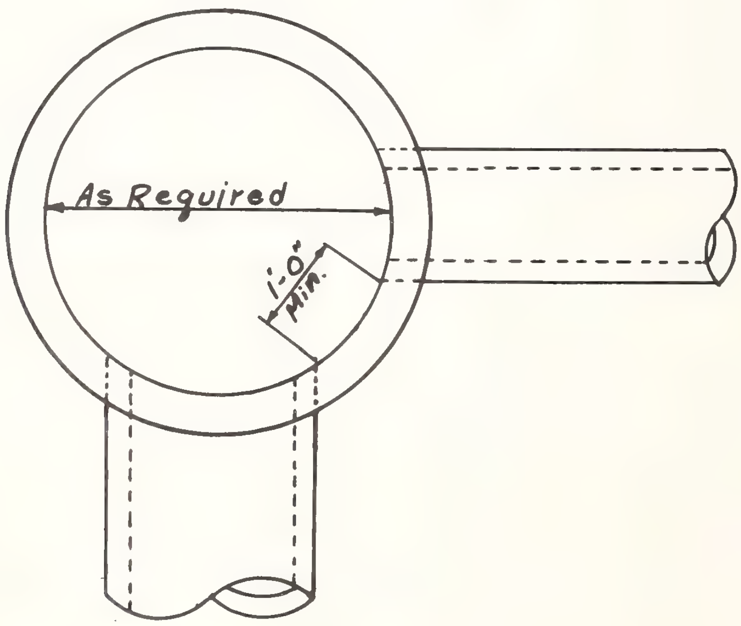


Figure 4.49

With the pipe diameter, slope, and velocity for a section of pipe known, the invert elevations for each end of the section may be established. This procedure is then repeated for the next downstream section.

Design Outfall - The purpose of the storm sewer outfall is to transport the storm water to a natural drainage and discharge it with as little erosion and pollution as possible. A storm sewer outfall consists of the outfall line, a treatment basin, and provisions for energy dissipation.

All design procedures for pipe type, size, and slope used for the collector system should be applied to the outfall. Open ditches for outfall lines should be investigated and used whenever possible. Storage in pipes and ditches is considered to be zero and therefore the maximum discharge determined for the sewer shall be used for sizing the outfall.

Whenever storm water is discharged into a stream containing fish life or used for drinking water, it is necessary to provide a treatment basin to minimize pollution to the receiving stream. The purpose of this basin is twofold. First, the solids such as sand and gravel should be settled out and, secondly, all floating contaminants such as gas and oil should be prevented from reaching the receiving stream. Figure 4.50 shows a typical treatment basin.

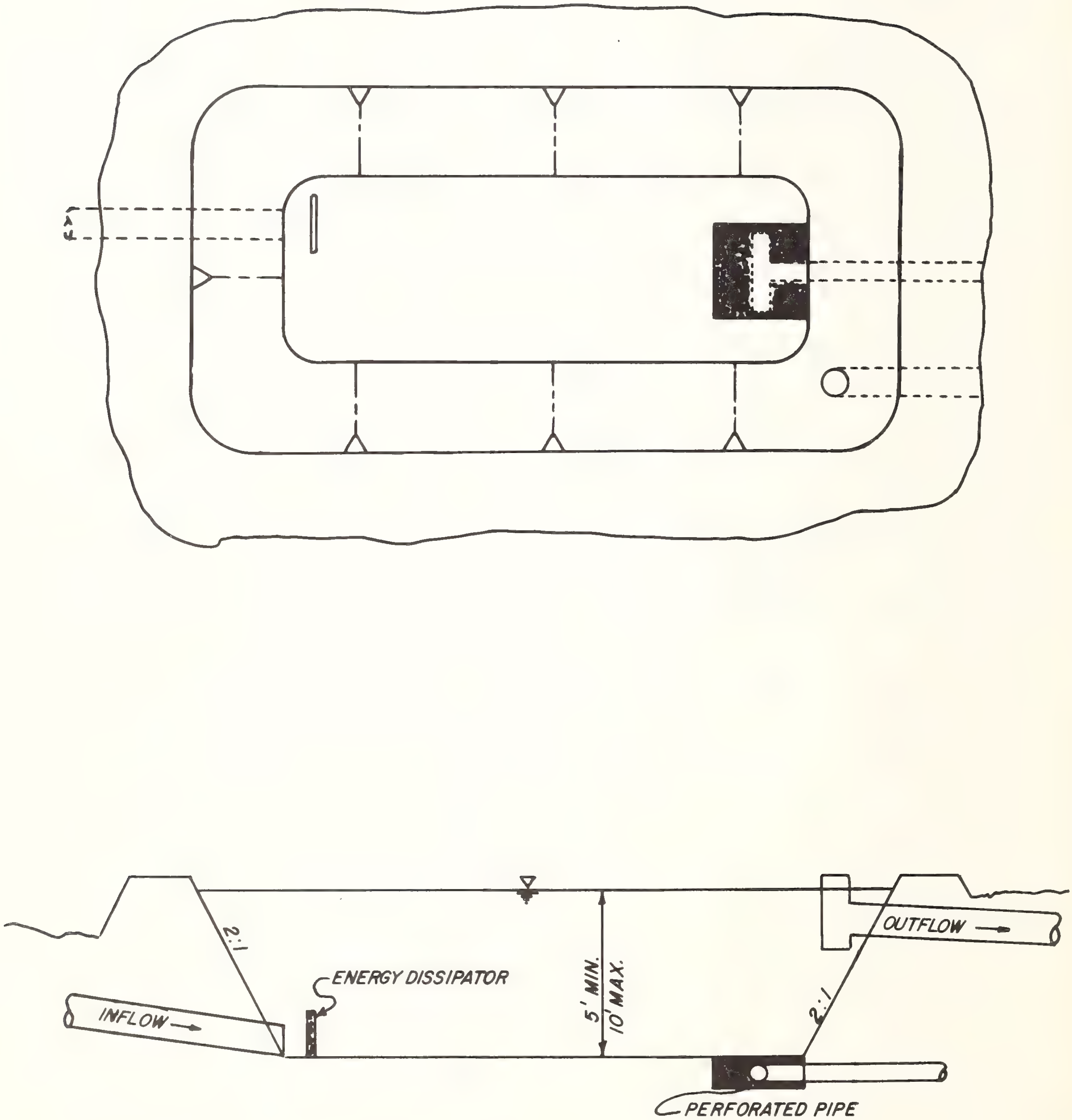


Figure 4.50
TYPICAL TREATMENT BASIN

The volume of the treatment basin is determined using the theory of sedimentation. The required volume is dependent upon the size of particles to be settled out, the depth of the pond and pond performance. The particle size to use for design can be determined from Table 4.36.

Table 4.36

Particle	Grain Diameter
Gravel	greater than 2.0 mm
Coarse Sand	2.0 mm to 0.42 mm
Fine Sand	0.42 mm to 0.074 mm
Silt	0.074mm to 0.005 mm
Clay	smaller than .005 mm
Colloids	smaller than .001 mm

The settling velocity of the design particle size in quiescent water can be determined from Figure 4.51. S_s is the specific gravity of the particle (usually S_s = 2.5).

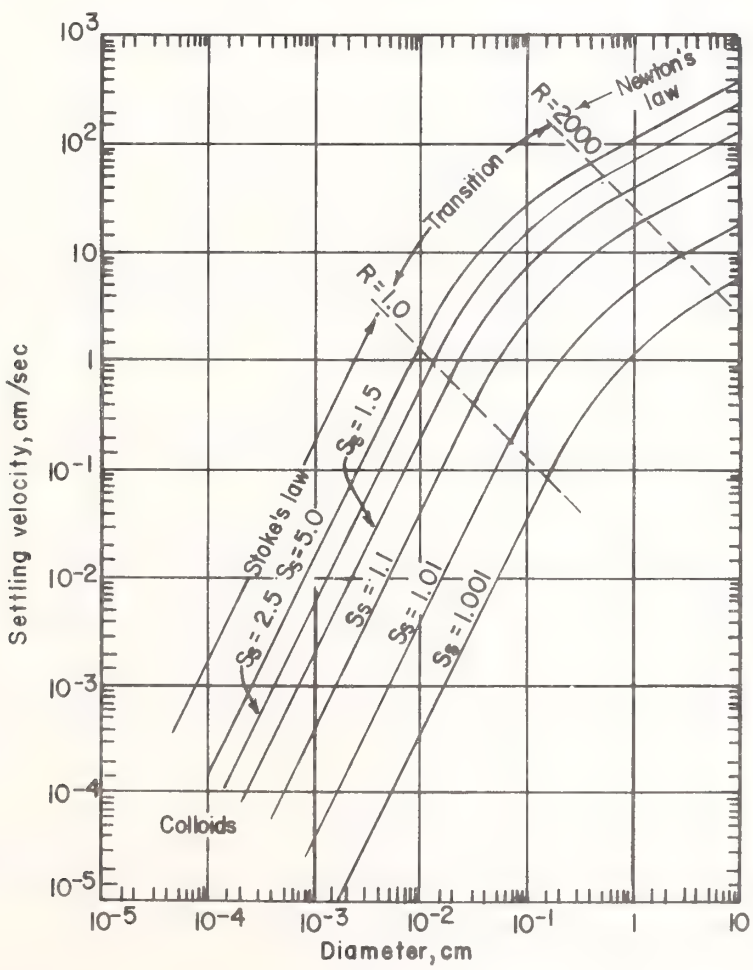


Figure 4.51
Settling Velocities of Discrete Spherical Particles in Quiescent Water at 50 Degrees F.

The depth of the proposed pond is divided by the settling velocity to determine the retention time required. The retention time is then multiplied by the design flow rate to determine the volume of the settling basin. This is the volume required to settle out 100 percent of the particles larger than the design particle if quiescent water conditions exist in the settling basin. However, currents reduce basin efficiency and the volume must be adjusted based upon the basin performance. Figure 4.52 gives adjustment factors based on percent removal and basin performance which the volume should be adjusted by to obtain the actual required volume of the settling basin.

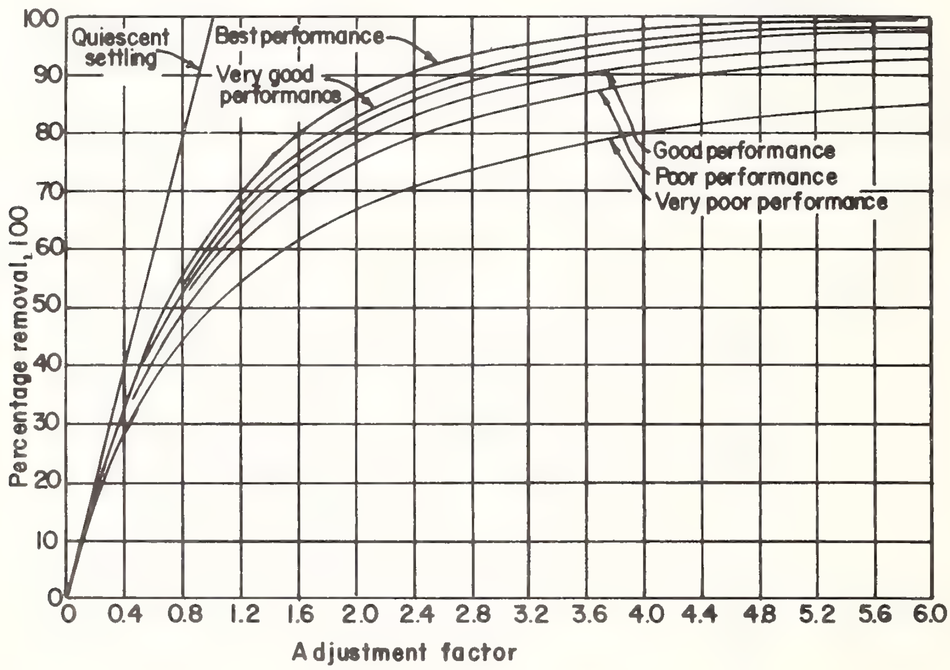


Figure 4.52
Performance Curves for Settling Basins of Varying Effectiveness

The width to length to depth ratio of the treatment basin should be established in such a manner as to keep currents and "short circuiting" to a minimum. The land available and ground water may also affect the basin dimensions.

Some means of draining the basin after a storm has stopped should be provided if natural seepage is inadequate.

The outlet of the basin should be designed to keep floating pollutants

from entering the receiving stream. The capacity of the outlet should be as great as the maximum capacity of the inlet.

The need for energy dissipators at the point the outfall enters the treatment basin and at the point it enters the receiving stream should be investigated. Section 4.8 discusses energy dissipators and erosion control.

4.72 INLET CAPACITY COMPUTER PROGRAM

The following program can be used to determine the capacity of a slotted drain.

Name: SDC Language: PLI Input Format: Run

Purpose: Determines Required Length and Size of Slotted Drain

Required Input Data: Gutter flow in cfs, the slope of the gutter, Manning's N for the gutter, and the street crown (1:X).

Abstract: This program solves the orifice and weir flow equations for varying flow depths to determine how much water will flow into the slotted drain. The program assumes orifice flow for gutter depths greater than .1 foot and weir flow for depths less than .1 foot. The program determines the length of slotted drain necessary to pick up the given flow for the given gutter conditions. The program also determines the size of slotted pipe required to carry the given flow on the given slope.

Limitations: The program is valid only for slotted drain placed next to the curb. When more than 40 feet of drain is required, the program prints the flow that 40 feet of drain will pick up. The program uses Manning's equation and corrugated steel pipe with 2 2/3 x 1 2/3 helical corrugations flowing full to determine the size of slotted pipe.

Sample Run:

```
      edit sdc basic
EDIT


      STA25+89RT
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   5,.02,.013,50
GUTTER FLOW =  5.0 CFS, DEPTH = .21 FT, VELOCITY =  4.3 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 28 FT
SIZE OF SLOTTED PIPE REQUIRED IS 12 INCH


EDIT  STA 34+38 lt
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   15,.02,.013,50
GUTTER FLOW = 15.0 CFS, DEPTH = .32 FT, VELOCITY =  5.7 FT/SEC
40 FT. OF SLOTTED DRAIN WILL PICK UP 12.9 CFS


EDIT
```

4.73 DESIGN EXAMPLE

The following design example is provided to show the general design approach and some of the details that must be considered when designing a storm sewer. The design will proceed using the step by step approach outlined previously. This example is the design of the storm sewer for project S-431 which is the reconstruction of Main Street in Hamilton, Montana. Some aspects of this project have been changed to make it a better example.

Determine the General Drainage Patterns - Figure 4.53 shows the street drainage patterns. From this Figure it appears that storm water in this area flows generally in a northerly direction. The storm sewer will pick up storm runoff from the residential area south of Main Street. The obvious solution is to carry the storm sewer along Main Street and discharge it into the Bitterroot River. Land is available near the river for construction of a settling basin. There is an existing storm sewer which runs down Sixth Street which will be discharged into the proposed storm sewer.

Determine the Design Storm - The storm sewer will serve a primarily residential area. Since the possibility of flood damage is quite remote, a design frequency of 2 years will be used.

The design storm is determined by referring to Section 3.5, Rational Method of Runoff Prediction. From Figure 3.7 Hamilton is found to be in Area I and it's 2-year 24-hour precipitation is 1.2 inches. This is an intensity of 1.2/24 = .05 inches per hour. Figure 3.8 is entered with this intensity and the design storm is identified. The design storm is shown on Figure 4.54

Locate Inlets and Establish Drainage Sections - Using Figure 4.53 to determine where it will be necessary to pick up runoff, the location of all inlets can be established. The location of these inlets are shown on the example plan sheets. All inlets will be corrugated steel slotted drain inlets

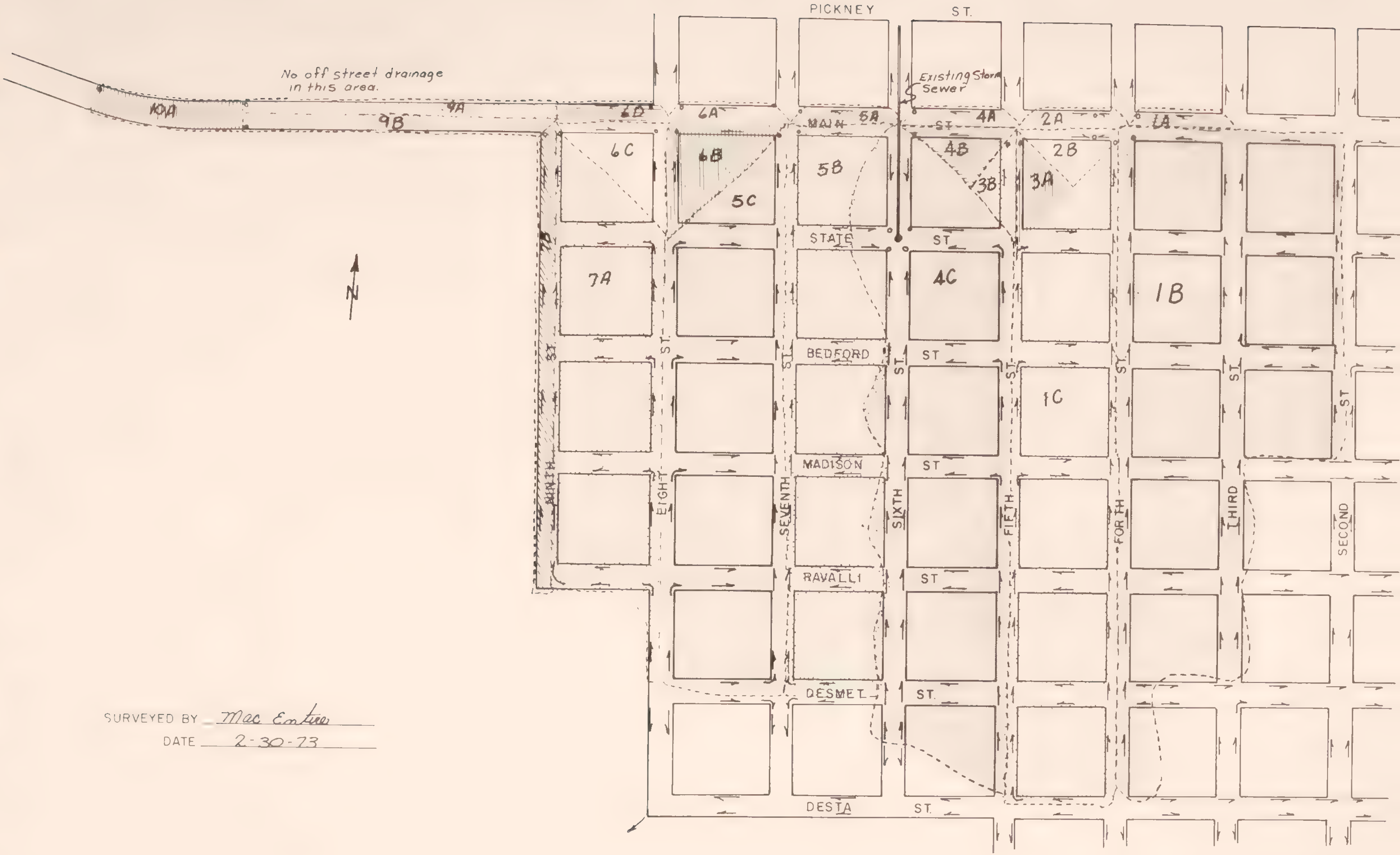


FIGURE 4.53
DRAINAGE PATTERNS

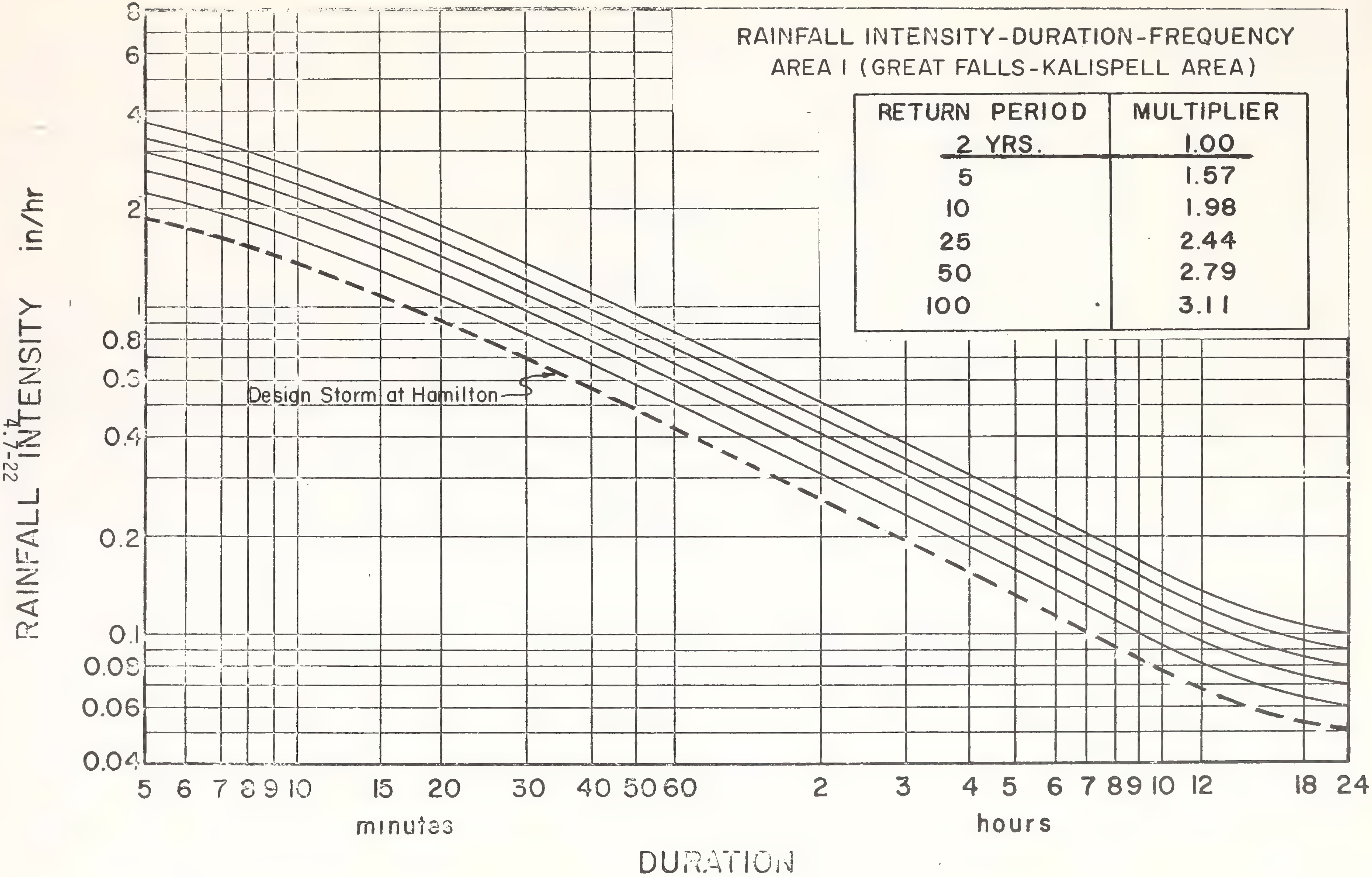


Figure 4.54 Rainfall Intensity-Duration-Frequecy Curve - Hamilton

to minimize the conflicts with bicycles with the exception of the inlets left and right of Station 63+50. Because these two inlets will be placed in a sag, the standard drop inlet with the grate bars transverse to the curb will be used. The inlets for section 4C are already in place.

With the inlet location established, the area which drains to each inlet is delineated. This is done on Figure 4.53 the drainage pattern map, with different shades.

<u>Determine Inlet Time and Runoff for Each Drainage Section</u> - The drainage area for each section is either calculated or planimetered depending on the section's shape. The runoff coefficient is estimated using Table 3.4 of the Hydrology Section. Most of the area drained by the proposed storm sewer is residential so a runoff coefficient of .3 will be used. For those sections which will drain just the highway or street, a coefficient of .9 will be used. The inlet time for each section is determined as outlined in Section 3.53 and using this inlet time an intensity is determined from Figure 4.54 The runoff for each section is then calculated using the Rational Formula. All of these values are tabulated. See Table 4.37.

Table 4.37

Inlet	Area Acres	Runoff Coefficient	Inlet Time Minutes	Intensity in./hr.	Runoff cfs
1A	.25	.90	5	1.8	.4
1B	27.7	.30	19	.9	7.5
1C	18.4	.30	20	.9	5.0
2A	.25	.90	5	1.8	.4
2B	.93	.30	5	1.8	.5
3A	.93	.30	5	1.8	.5
3B	.79	.30	5	1.8	.4
4A	.25	.90	5	1.8	.4
4B	1.1	.30	5	1.8	.6
4C	20.7	.30	18	.9	5.6
5A	.25	.90	5	1.8	.4
5B	12.8	.30	15	1.1	4.2
5C	15.3	.30	18	.9	4.1
6A	.25	.90	5	1.8	.4
6B	2.1	.30	5	1.8	1.1
6C	1.7	.30	5	1.8	.9
6D	.25	.90	5	1.8	.4
7A	12.3	.30	19	.9	3.3
7B	1.4	.9	13	1.3	1.6
9A	.73	.9	8	1.5	1.0
9B	.73	.9	8	1.5	1.0
10A	.67	.9	5	1.8	1.1

The inlets and inlet to manhole connectors are sized using these runoff values. Using computer program "SDC Basic" it is determined that 10 feet of slotted drain will be sufficient for all of the inlets except inlets 1B, 5B, 5C, and 7A which will require 20 feet of slotted drain. The inlet to manhole connectors cannot be sized until the invert elevations of the manholes are determined.

Establish Preliminary Storm Sewer Layout - The typical section for Main Street includes an 18 foot painted median. Since there will be inlets on both sides of the street, the logical location for the sewer line is in the median. Manholes will be placed at each intersection, at Station 63+50 to pick up flow from the drop inlets, and at Station 40+86 so the interval between manholes does not exceed 500 feet. The location of the sewer line and manholes are shown on the example plan sheets.

```
      Inlet 1B on 4th St.
READY edit sdc basic
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   3.6,.003,.015,50
GUTTER FLOW =  3.6 CFS, DEPTH = .29 FT, VELOCITY =  1.8 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 14 FT
SIZE OF SLOTTED PIPE REQUIRED IS 18 INCH


EDIT  Inlet 1B on Main St.
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   3.7,.01,.015,50
GUTTER FLOW =  3.7 CFS, DEPTH = .23 FT, VELOCITY =  2.8 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 18 FT
SIZE OF SLOTTED PIPE REQUIRED IS 12 INCH


EDIT  Inlet 1C
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   5,.003,.015,50
GUTTER FLOW =  5.0 CFS, DEPTH = .32 FT, VELOCITY =  1.9 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 18 FT
SIZE OF SLOTTED PIPE REQUIRED IS 18 INCH


EDIT  Inlet 5B
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   4.2,.003,.015,50
GUTTER FLOW =  4.2 CFS, DEPTH = .30 FT, VELOCITY =  1.8 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 16 FT
SIZE OF SLOTTED PIPE REQUIRED IS 18 INCH


EDIT  Inlet 5C
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   4.1,.003,.015,50
GUTTER FLOW =  4.1 CFS, DEPTH = .30 FT, VELOCITY =  1.8 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 16 FT
SIZE OF SLOTTED PIPE REQUIRED IS 18 INCH


EDIT  Inlet 7A
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   3.3,.003,.015,50
GUTTER FLOW =  3.3 CFS, DEPTH = .28 FT, VELOCITY =  1.7 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED = 13 FT
SIZE OF SLOTTED PIPE REQUIRED IS 15 INCH


EDIT  Inlet 7B
EDIT  run
INPUT GUTTER FLOW,GUTTER SLOPE, MANNINGS N, CROWN 1:X
  ?   1.6,.003,.015,50
GUTTER FLOW =  1.6 CFS, DEPTH = .21 FT, VELOCITY =  1.4 FT/SEC
LENGTH OF SLOTTED DRAIN REQUIRED =  8 FT
SIZE OF SLOTTED PIPE REQUIRED IS 12 INCH
```

Establish Profile With Control Elevations - All control elevations are shown on the plan sheets. The profile shown is the finished grade profile immediately over the pipe and the natural ground over the outfall. If necessary, the 4 inch water mains at each intersection may be raised or lowered to avoid our storm sewer. The invert elevation of Manhole #4 must be 3554.0 or less so that the exising 24 inch concrete storm sewer may be intercepted. The 15 inch sanitary sewer crossing at Station 49+71 will also have to be avoided. This sewer is shown on the profile. The 10 inch water main left of centerline is approximately 6 feet deep and should not affect our inlets on that side. The outlet control elevations are also shown on the plans. The mean annual flood elevation is 3537.2 and the 50 year flood elevation is 3540.4.

Determine Required Capacity, Pipe Diameter, Slope and Velocity of Each Pipe Section - The following Storm Sewer Design Sheet shows these computations. Weighted runoff coefficients $\left(\frac{C_1 A_1 + C_2 A_2}{A_1 + A_2}\right)$ are used as some of the inlets draining to one manhole have different values. The manhole sizes are determined graphically. The inlet to manhole connectors are also sized now.

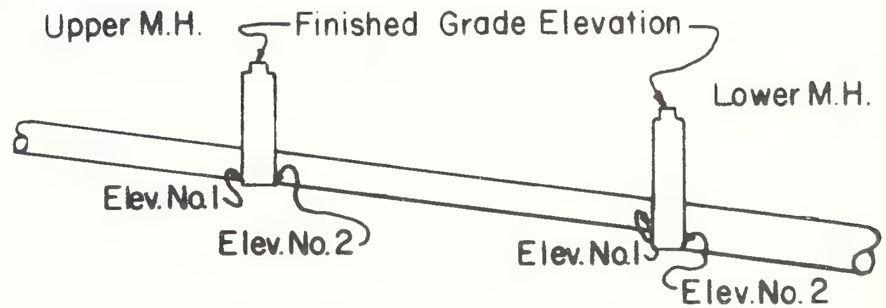
Design Outfall - The outfall line for the storm sewer will be to the right of centerline and will include a treatment basin. For design purposes it will be assumed that the river will be at mean annual flood stage during our design storm.

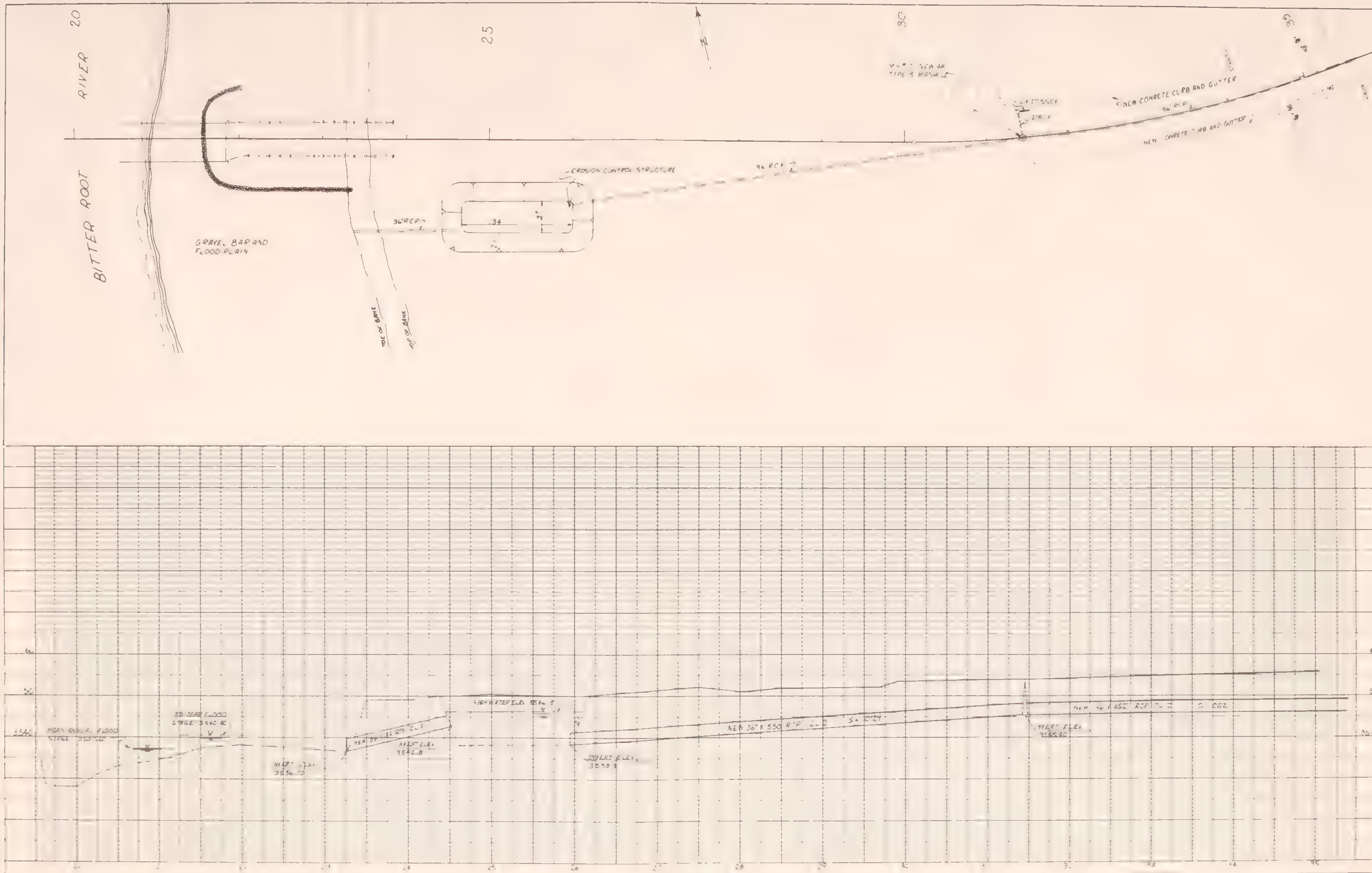
The line from Manhole #10 to the treatment basin is designed using the hydraulic grade line rather than the invert slope. The required hydraulic gradient for a 36" RCP carrying 29.4 cfs is approximately .0025. The flowline elevation in Manhole #10 is approximately 3547.7 and the length of this outfall line is approximately 550. Therefore, the maximum treatment pond elevations should be about 3547.7 - (.0025 x 550) = 3546.3. Using a depth in the pond of about 8 feet establishes the elevation of the bottom at 3538.3.

STORM SEWER DESIGN SHEET

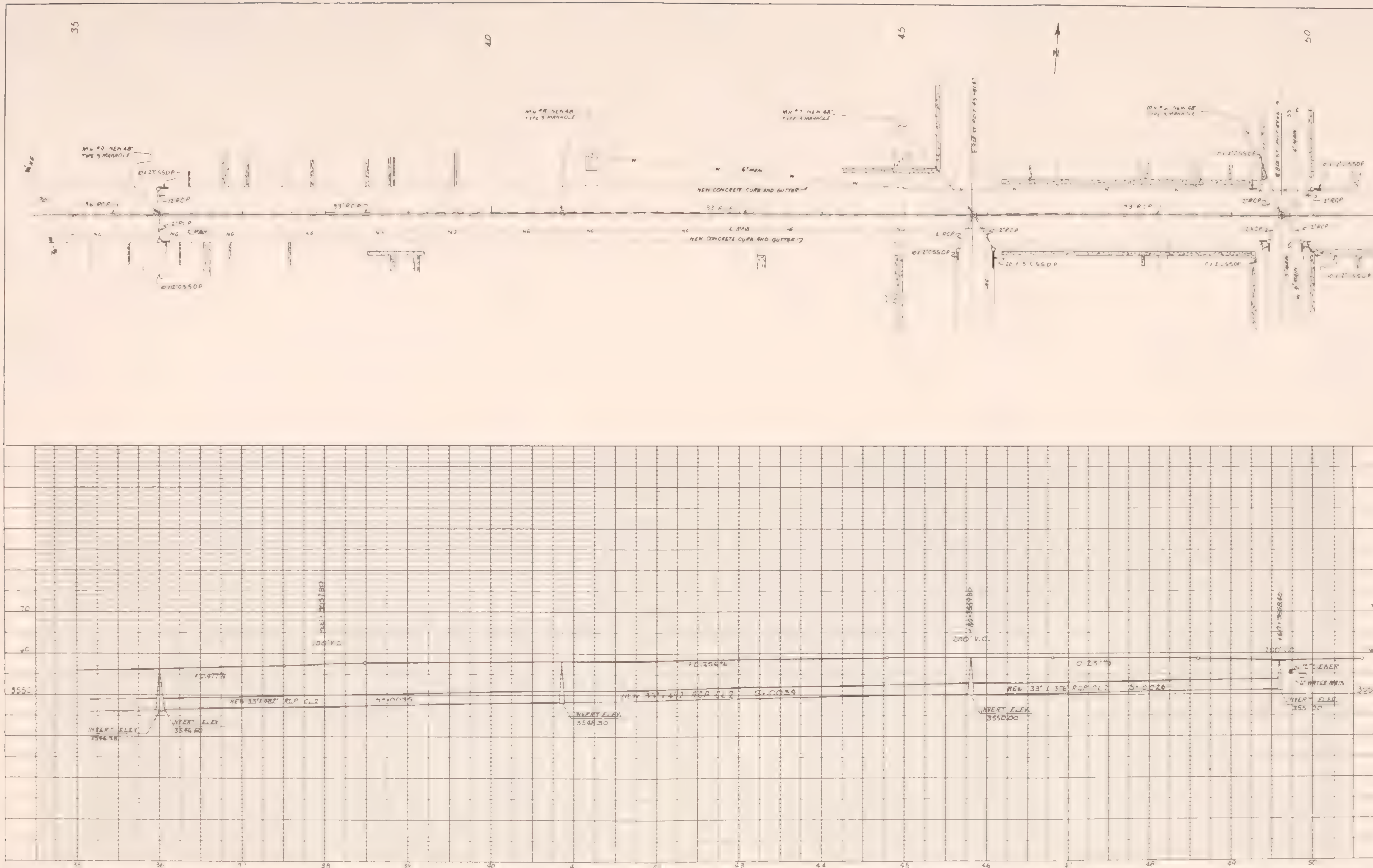
N = .012
DESIGN FREQ. 2 YRS.
DESIGNER Hi Drawlics
DATE 3-27-74
PROJECT S-431(3)
Hamilton South

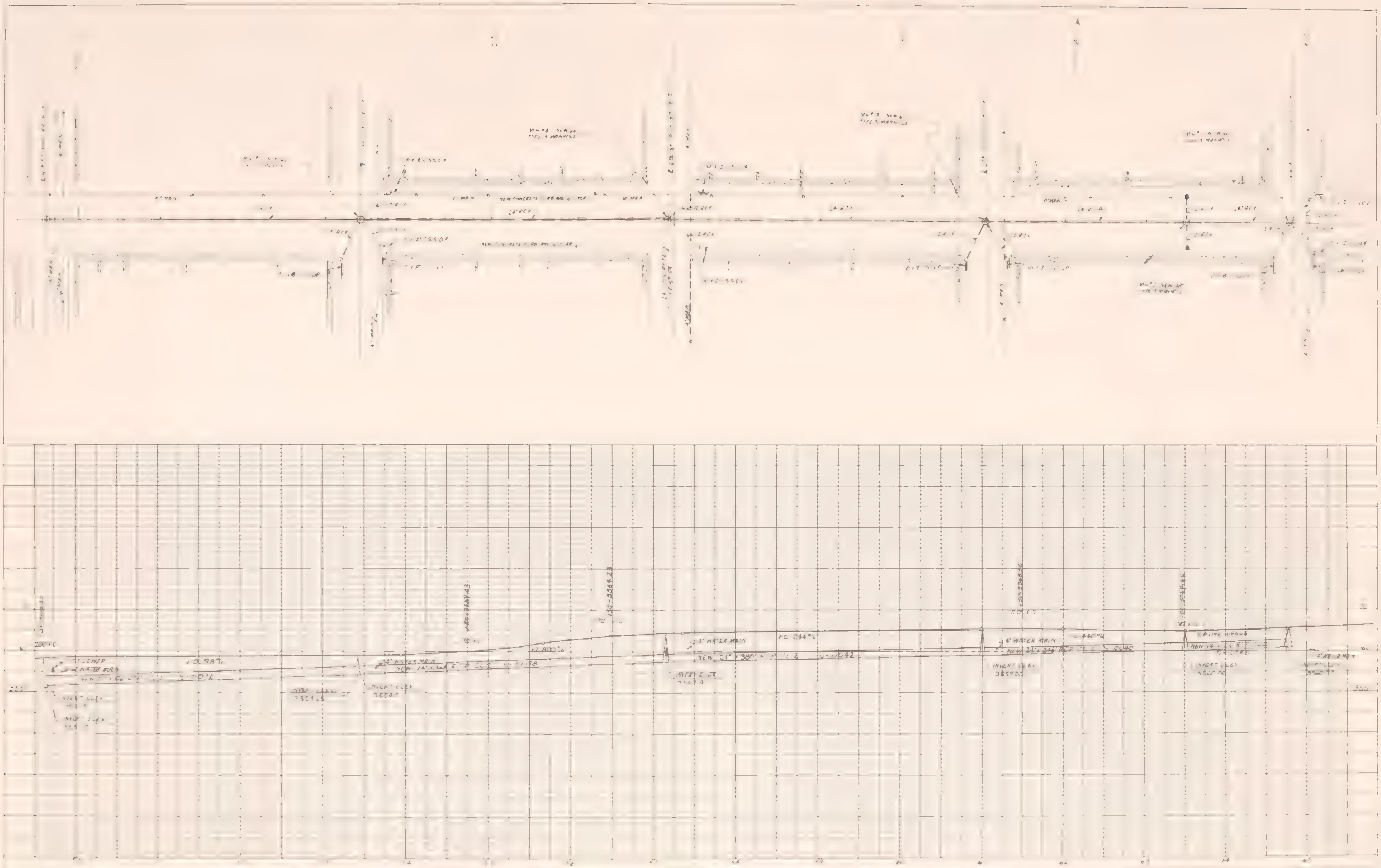
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
LINE NO.	LOCATION	STATION	INCR. AREA "A"	"C"	INCR. CxA	TOTAL C x A	TIME OF CONC. MIN.	I IN./HR	Q C.F.S.	PIPE SLOPE FT./FT	DIA. OF PIPE INS.	V FT/SEC	L FT.	TIME OF FLOW MIN.	CAP. OF SEWER C.F.S.	FINISHED GRADE ELEVATION AT MANHOLE	INVERT ELEVATIONS ELEV. NO. 1	INVERT ELEVATIONS ELEV. NO. 2	M.H. DIA.	
1	4th St.	64+78	46.4	.30	13.9	13.9	20	.9	12.5	.0055	24	6.1	128	.3	19	65.8	–	60.7	60"	
2		63+50	1.2	.43	.5	14.4	20.3	.9	13.0	.004	24	5.5	250	.7	16	64.8	60.0	60.0	48"	
3	5th St.	61+00	1.7	.30	.5	14.9	21.0	.9	13.4	.0042	24	5.5	385	1.1	16	65.6	59.0	59.0	60"	
4	6th St.	57+15	22.1	.31	6.9	21.8	22.1	.9	19.9	.0078	24	7.8	373	.8	23	64.6	57.4	57.4	66"	
5	7th St.	53+42	28.4	.30	8.5	30.3	22.9	.8	24.2	.0072	27	8.2	380	.8	29	59.0	54.5	54.25	66"	
6	8th St.	49+62	4.3	.37	1.6	31.9	23.7	.8	25.5	.0026	33	5.8	380	1.1	30	58.3	51.5	51.0	48"	
7	9th St.	45+82	13.7	.36	4.9	36.8	24.8	.8	29.4	.0034	33	6.3	496	1.3	34	59.2	50.0	50.0	48"	
8		40+86	0	–	0	36.8	26.1	.7	29.4	.0035	33	6.5	486	1.2	35	58.2	48.3	48.3	48"	
9		36+00	1.5	.9	1.4	38.2	27.3	.7	29.4	.0021	36	5.5	455	1.4	35	56.3	46.6	46.35	48"	
10		31+45	.7	.9	.6	38.8	28.7	.7	29.4	Hydraulic Gradient .0025	36	–	550	–	38	54.2	45.4	–	48"	
	Treatment Basin	77' Lt. of 26+00	–	–	–	–	–	–	–											





BITTER ROOT RIVER
GRAVEL BAR AND FLOOD PLAIN
TOE OF BANK
TOP OF BANK
EROSION CONTROL STRUCTURE
36" RCP
NEW CONCRETE CURB AND GUTTER
20
25
30
35
N


35
40
45
50
NEW CONCRETE CURB AND GUTTER
NEW CONCRETE CURB AND GUTTER
TYPE 3 MANHOLE
33" RCP
100' V.C.
200' V.C.
200' V.C.
INVERT ELEV.
3546.38
INVERT ELEV.
3546.60
INVERT ELEV.
3548.50
INVERT ELEV.
3550.20
INVERT ELEV.
3550
70
60
3550
36
38
39
40
41
42
43
44
45
46
48
49
50


8" WATER MAIN
INLET ELEV

The pond will be sized to settle out fine sand particles with diameters of .074 mm. From Figure 4.51 the settling velocity of these particles is .4 cm/sec or .013 ft/sec. The time for one of these particles to settle the 8 foot depth is 8/.013 = 615 sec. This time is used as the required retention time then the required volume of the treatment basin is 615 x 29.4 = 18,080 ft^3 if quiescent settling conditions are assumed. From Figure 4.52, assuming good pond performance and 90% removal, an adjustment factor of 3.5 is chosen. This makes the required pond volume 18,080 x 3.5 = 63,280 $ft.^3$. A 37' x 134' pond with 2:1 side slopes will be used.

The outlet from the treatment basin to the river will be designed for 38 cfs which is the maximum capacity of the line into the basin. The head water elevation will be 3546.3. Using inlet control and an entrance coefficient of .9, because of the inlet configuration required to keep floating pollutants out of the river, the required head is approximately 3.5 for a 36" RCP. This makes the invert elevations 3546.3 - 3.5 = 3542.8.

The soil survey indicates there is a coarse gravel - sandy material in this area and that the pond will drain by itself after the storm stops. Also, an erosion control structure will be required at the inlet to the basin.

References

1. Design of Storm - Water Inlets, Report of the Storm Drainage Research Committee of the Storm Drainage Research Project, John Hopkins University, Baltimore, Maryland, June. 1956.

2. Dodge, E.R., Final Report Application Hydrologic and Hydraulic Research to Culvert Selection in Montana, Volume I and II, 1972, for the Montana Department of Highways, in cooperation with U.S. Department of Transportation, Federal Highway Administration.

3. Fair, G.M., J.C. Geyer, and D. A. Okum, Water Purification and Wastewater Treatment and Disposal, John Wiley and Sons, Inc. New York, 1968.

4. Harden, E.E., Urban Storm Sewer Design for Idaho Highways, for the Department of Highways, State of Idaho.

5. Hydraulic Engineering Circular No. 12, Drainage of Highways Pavements, U.S. Department of Transportation, Federal Highway Administration, March, 1969.

6. Linsley, R.K. and J.B. Franzini, Water Resources Engineering, McGraw-Hill Book Company, New York, 1964.

7. Urban Storm Drainage Criterial Manual, Volume 1 and 2, prepared for Denver Regional Council of Governments by Wright-McLaughlin Engineers, Engineering Consultants, Denver, Colorado, March, 1969.

8. Woo, D.C. and J.S. Jones, Hydraulic Characteristics of Two Bicycle - Safe Grates A Preliminary Report, Environmental Design and Control Division, Offices of Research and Development, Federal Highway Administration, Washington, D.C., November, 1973.

SECTION 4.8
EROSION CONTROL

4.8 EROSION CONTROL

Introduction

The power of moving water is impressive, especially when erosion such as in the Grand Canyon is considered. The same erosive force, to a lesser extent, can damage or destroy unprotected soil surfaces. Water moving over a soil surface causes erosion that often results in two undesirable consequences - damage to the soil surface, and silting or the depositing of the eroded soil in another waterway, such as a stream or a lake. Natural waterway channels are generally stabilized due to the action of erosion over long periods of time, and much of the earth's land area is protected from erosion by vegetation cover. When highway construction interferes with the natural flow of water, drainage channels must be designed and built to redirect the water to a natural waterway. Highway drainage channels should be made erosion resistant to prevent unsightly gullies, keep maintenance costs down, prevent silting of some other waterway or body of water, and to prevent damage to the highway.

This section deals with erosion prevention associated with waterway channels including discussions and design proceedures for rip rap and filter blankets, ditch linings, spur dikes, and energy disapators.

4.81 RIP RAP AND FILTER BLANKETS

Rip Rap Design

The rip rap design procedure presented here should only be used when designing rip rap for channels where flow is markedly non-uniform or when discontinuities such as bridge abutments disrupt the flow. When using rip rap as a channel lining the design procedures presented in Section 4.82 should be used. The procedures presented here are taken from Hydraulic Engineering Circular No. 11.

Types of Rip Rap - The types of rip rap slope protection discussed in this section are:

1. Random rip rap
2. Hand laid rip rap
3. Wire - enclosed rip rap
4. Grouted rip rap
5. Sacked concrete rip rap
6. Concrete - slab slope protection

1. Random rip rap is graded stone dumped on a prepared slope in such a manner that segregation will not take place. Dumped stone rip rap is the most flexible of the types considered here and will adjust itself to uneven bank settlement. In most areas random rip rap is the least costly type.

2. Hand-placed rip rap is stone laid carefully by hand or by derrick following a more or less definite pattern with the voids between the larger stone filled with smaller stone and the surface kept relatively even. The resulting protection approaches good dry rubble in quality and appearance, but this type of rip rap is rigid and lacks the strength necessary to bridge even minor movement of the surface which it protects.

3. Wire-enclosed rip rap is stone placed in wire baskets or in wire covered mats. Wire - enclosed rip rap is generally used because rock of

suitable size is not available. This rip rap is effective until the wire enclosure fails.

4. Grouted rip rap is rip rap with the interstices filled with portland cement mortar. The use of grouted rip rap is seldom justifiable when stone of suitable size is available.

5. Concrete rip rap in bags is concrete in suitable burlap bags that are hand placed in contact with adjacent bags.

6. Concrete-slab rip rap is plain or reinforced concrete slabs poured or placed on the surface to be protected.

Random Rip Rap Design - The resistance of random rip rap to displacement by moving water depends upon:

1. Weight, size, shape, and composition of the individual stones
2. The gradation of the stone
3. The depth of water over the stone blanket
4. The steepness and stability of the protected slope and angle of repose of rip rap
5. The stability and effectiveness of the filter blanket on which the stone is placed
6. The velocity of the flowing water against the stone
7. The protection of toe and terminals of the stone blanket

The size of stone needed to protect a streambank or highway embankment from erosion by a current moving parallel to the embankment is determined by the use of Figure 4.55 and 4.56. Size (k) is the diameter, in feet, of a spherical stone that would have the same weight as the 50 percent size of stone. The size of stone is found by a trial-and-error procedure which consists of first estimating a stone size.

The mean velocity (V_m) of the stream during the design flood must then be converted to velocity against the stone by use of Figure 4.55. The ratio ($\frac{k}{d}$) of the equivalent spherical diameter of the 50 percent stone size to the depth of flow during the design flood is computed by using 0.4 of the total

depth when the depth of flow exceeds about 10 feet. The reason for this is that use of the total depth would result in a stone size which would be adequate at the total depth but which might be too light to provide protection near the water surface.

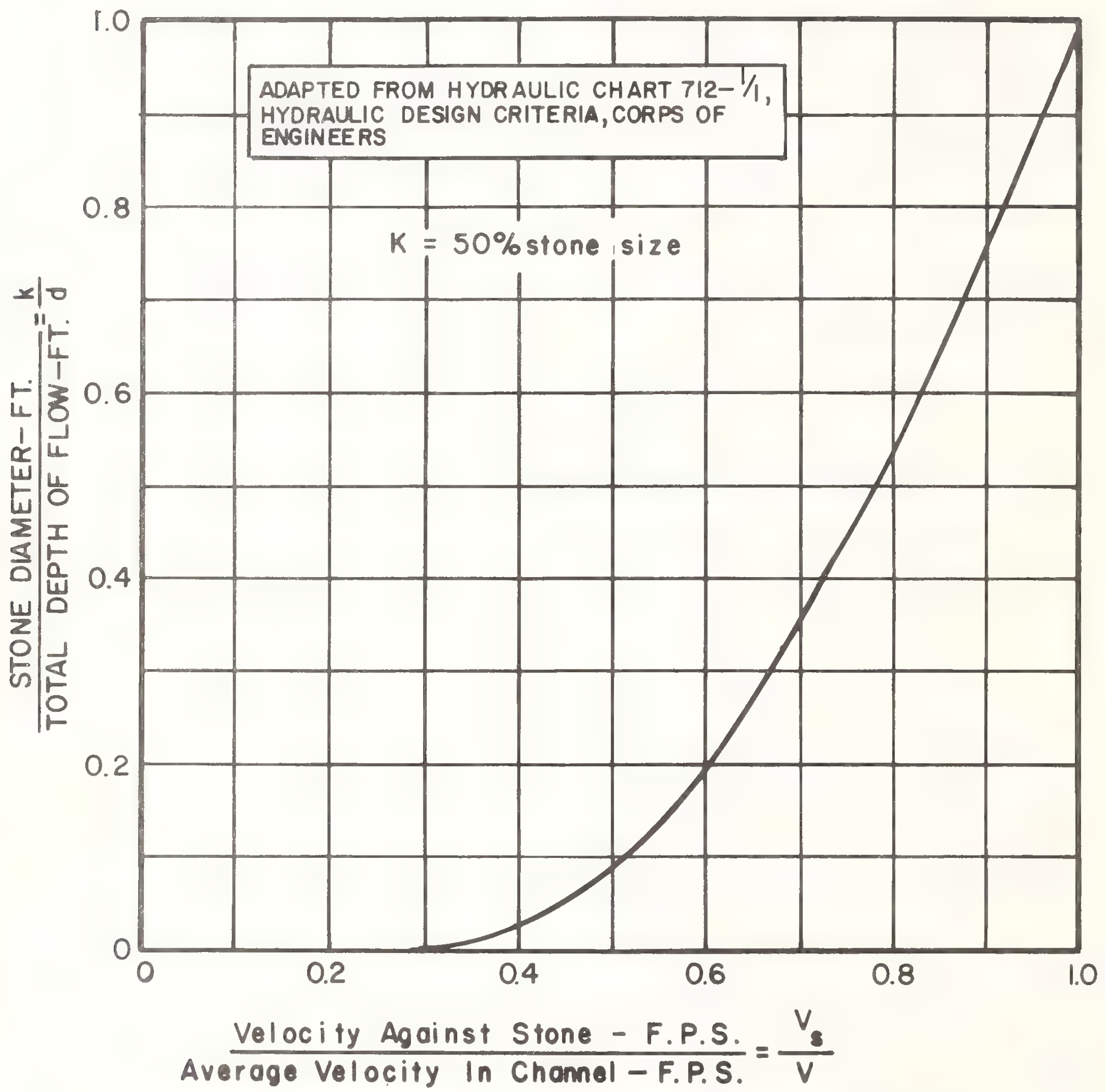


FIG. 4.55 - VELOCITY AGAINST STONE ON CHANNEL BOTTOM

With the velocity against the stone (V_S) enter Figure 4.56 and read the stone size for the embankment slope. The stone size from Figure 4.56 is the 50 percent (median) size, by weight, of a well graded mass of stone with a unit weight of 165 pounds per cubic foot. If the stone size from Figure 4.56 agrees with the assumed stone size, this is the correct size. If not, the procedure is repeated until the assumed size is in reasonable agreement with

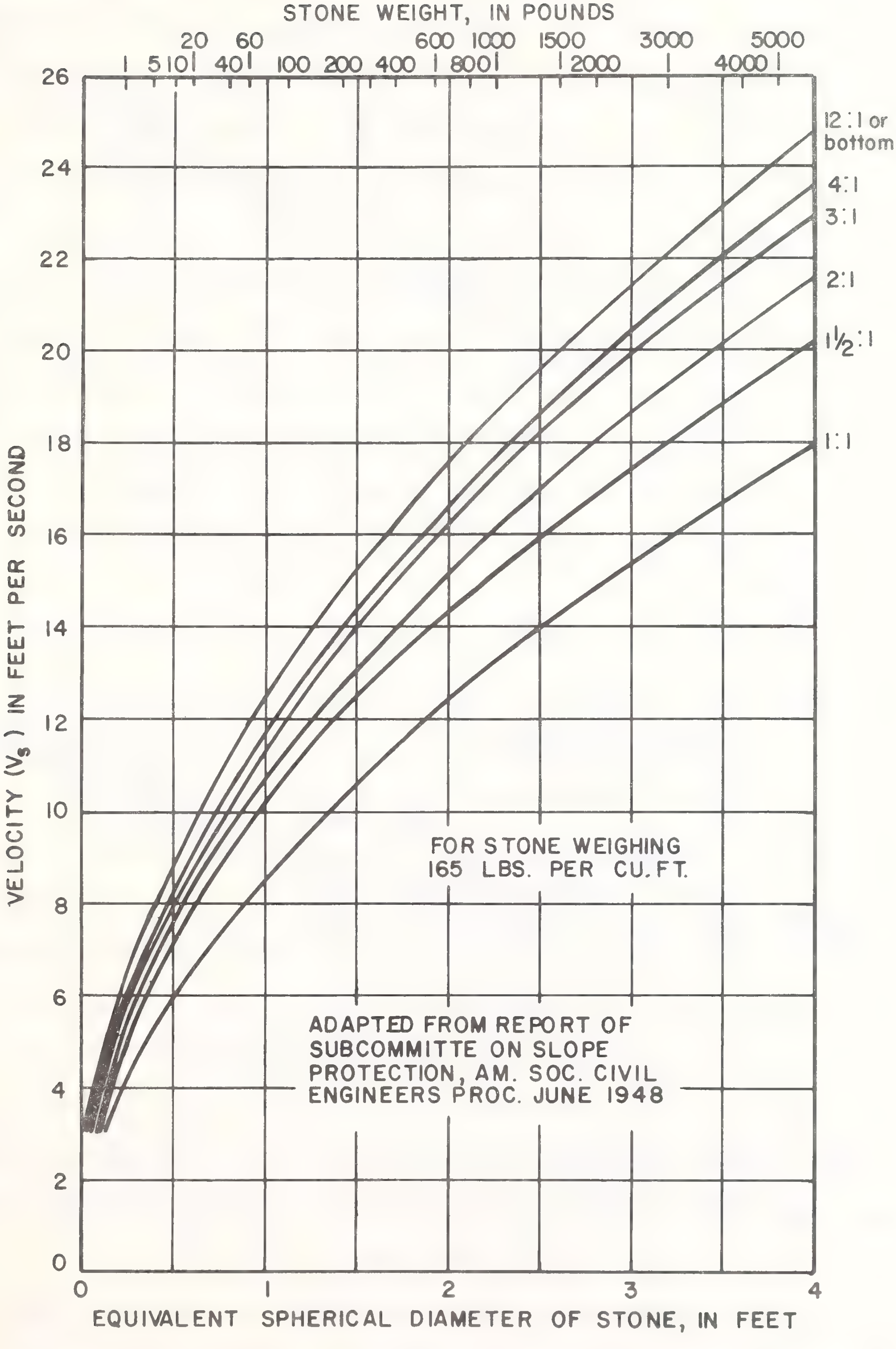


FIG. 4.56-SIZE OF STONE THAT WILL RESIST DISPLACEMENT FOR VARIOUS VELOCITIES AND SIDE SLOPES

the size from Figure 4.56.

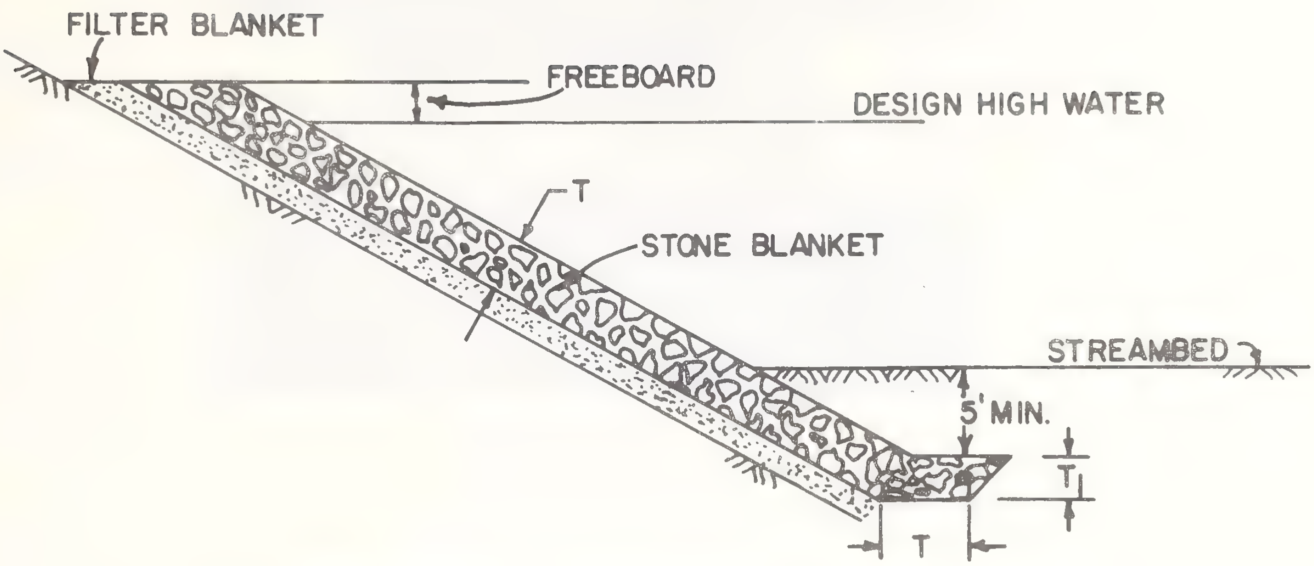
The size of stone required to resist displacement from direct impingement of the current as might occur with a sharp change in stream alignment is greater than the value obtained from Figure 4.55, although research data is lacking on just how much larger the stone should be. The California Division of Highways (6) recommends doubling the velocity against the stone as determined for straight alignment before entering Figure 4.56 for stone size.

Lane in his Design of Stable Channels - recommends reducing the allowable velocity by 22 percent for very sinuous channels; for determing stone size by Figure 4.56, the velocity (V_s) would be increased by 22 percent. Until data is available for determining the stone size at the point of impingement, a factor which would vary from 1 to 2 depending upon the severity of the attack by the current, should be supplied to the velocity V_s before entering Figure 4.56.

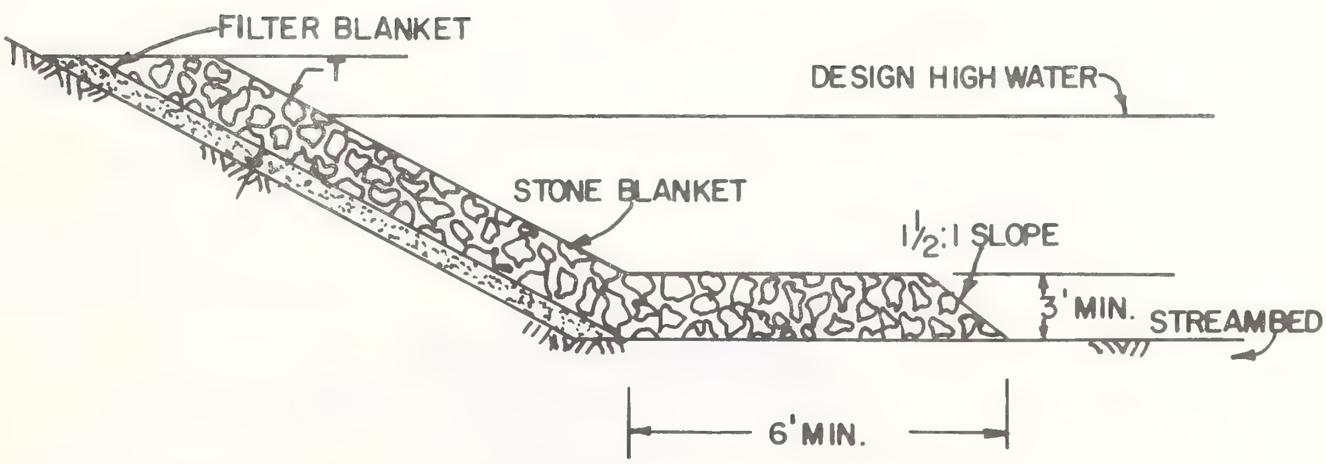
The upper vertical limit of the rip rap cover should extend above design high water. The allowance for freeboard depends upon the velocities near the rip rap cover and, at some locations, upon the height of waves that might be generated on the water surface. Established sod above the stone protection will provide considerable protection from floods which overtop the rip rap cover.

Where the stream channel is composed of sand or silt, bank protection should extend a minimum vertical distance of 5 feet below the streambed on a continuous slope with the embankment (Figure 4.57A). On the outside of curves or sharp bends, scour is particularly severe, and the toe of the bank protection should be placed deeper than in straight reaches. Where a toe trench cannot be dug, the rip rap blanket should terminte in a stone toe at the level of the streambed (Figure 4.57B). The toe provides material which will fall into a scour hole and thus extend the blanket.

On large rivers having a considerable depth of flow at low water stages, the Corps of Engineers carries the stone protection 5 feet vertically below



A – STONE BLANKET AND TOE TRENCH DETAIL



B – STONE BLANKET AND TOE DETAIL

FIG. 4.57 – TYPICAL SECTIONS, STONE SLOPE PROTECTION FOR TANGENT REACHES WITH EROSIVE BEDS

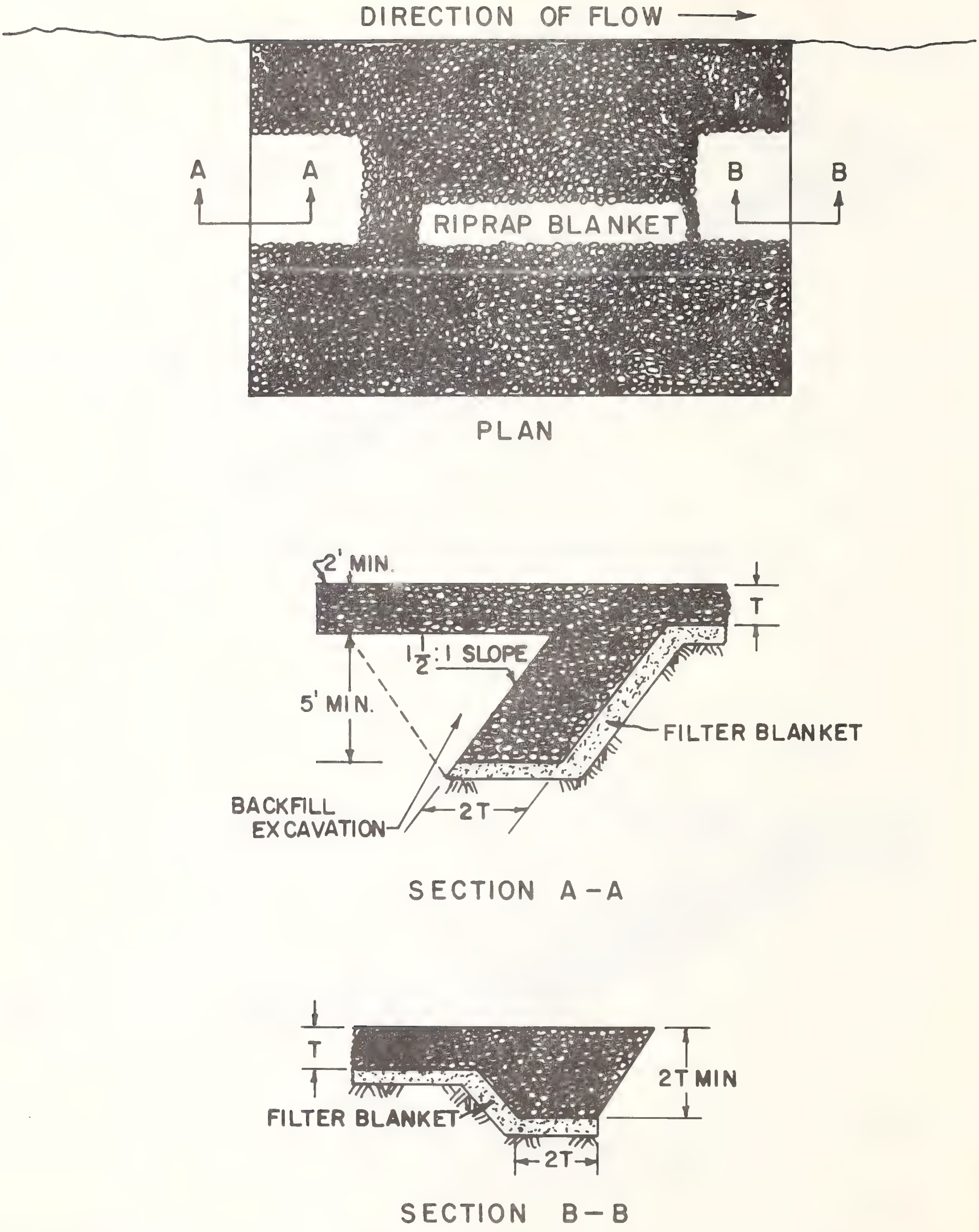


FIG.4.58 – DETAILS OF CUTOFF AT TERMINALS OF RIPRAP BLANKET

mean low water and omits the toe. The stone blanket should be keyed into a berm when a toe or toe trench is not provided. The purpose of the toe protection is to prevent undermining, not to support the blanket. Unless the protection has sufficient stability to support itself on the embankment slope, the protection cannot be considered adequate.

The bank protection should extend both upstream and downstream from the points of reverse curvature on the outside of a curved channel. Bank protection is usually not required on the inside of the curve unless return of overbank flow creates a scour problem. On a straight channel, bank protection should begin and end at a stable feature in the bank if practicable. Such features might be outcroppings of erosion resistant materials, trees, vegetation, or other evidence of stability. When a stable feature does not exist, cutoffs should be provided (Figure 4.58). If the protective cover is long, intermediate cutoffs might be required to reduce the hazard of complete failure of the stone blanket.

The thickness of the stone blanket should be at least equal to the maximum size stone.

The gradations of the three classes of random rip rap used by the Montana Department of Highways are listed below along with their characteristics needed for design.

<u>Hand Laid Rip Rap Design</u> - Hand laid rip rap was at one time considered superior to random rip rap, and both the size of stone and the thickness of the hand - placed stone blanket was specified as one-half that required for random stone. The supposed superiority of hand-placed rip rap was refuted by a performance survey of the majority of the large earth dams in the United States conducted by the Corps of Engineers in 1946. The survey showed that hand - placed rip rap was not as satisfactory as an equivalent thickness of random rip rap. The percentage of failures in hand - placed rip rap slope protection was six times that of random rip rap, and the percentage of failures

of concrete pavement used for slope protection was slightly over seven times that of random rip rap.

Random Rip Rap

Stone Size	Equivalent Spherical Diameter	Percent of Total Weight That Must be Smaller Than Given Size
	CLASS I (D_{50} = .66', D_{15} = .6', D_{85} = .9')	
100 lbs	1.05'	100%
60	.88'	70 - 90
25	.66'	40 - 60
2	.27'	0 - 10
	CLASS II (D_{50} = 1.32', D_{15} = .6', D_{85} = 1.6')	
700 lbs	2.00'	100%
500	1.79'	70 - 90
200	1.32'	40 - 60
20	.61	0 - 10
	CLASS III (D_{50} = 2.00', D_{15} = .8', D_{85} = 2.6')	
2000 lbs	2.82'	100%
1400	2.53'	70 - 90
700	2.00'	40 - 60
40	.77'	0 - 10

The Corps of Engineers opinion, was: "Hand - placed rip rap is not as satisfactory as an equivalent thickness of random rip rap, and a filter blanket underneath all rip rap is essential".

Hand - placed rip rap requires a much firmer support from the bank being protected than does random rip rap because it does not have the strength to resist nor the capability to adjust to movement of the supporting material. Hand - placed rip rap is particularly susceptible to damage from ice floating in the stream.

Except for method of placing and greater emphasis on firm support and protection of blanket edges, the discussion under random rip rap applies to hand - placed rip rap.

Wire Enclosed Rip Rap Design - The use of wire-enclosed rip rap is generally restricted to locations where the only rock economically available is too small for random rip rap. The design of wire-enclosed rip rap is somewhat arbitrary,

being dependent upon the size or rock available. The mesh size of the wire is also dependent upon the size of rock used for rip rap. Wire-enclosed rip rap has been used in some instances as toe protection for other types of rip rap. This type of protection is flexible to an extent, but the protection is limited to the life of the wire used for enclosing the stone. Wire containing .2% copper with a medium zinc coating has been found to have the longest life. California has found that wire-enclosed rip rap does not work well on curves where displacement might require a lengthening or shortening of the protection.

The wire baskets are first formed and then filled with stone. The baskets are tied together to form a mattress and anchored to the slope. On all designs the blanket should be divided into compartments so that one compartment can fail without losing all of the blanket.

Commercial baskets should be specified. They are available in a wide variety of shapes and sizes.

The discussion of the extent of the random rip rap blanket also applies to wire baskets.

<u>Grouted Rip Rap Design</u> - Grouted rip rap is used where stone of suitable size for other types of rip rap are not available. Wire can be embedded in the rip rap to increase the tensile strength of the protective cover. The finished protection is rigid and has little strength. For this reason, the embankment protected must provide adequate support and the edges of the rip rap cover must be protected from undermining at the toe and at the terminals. Design data are lacking for determining the specific thickness of the cover. The grouted rip rap may be left with a rough surface by brushing the grout until from one-fourth to one-half the depth of the stone is exposed.

Weep holes should be provided in the blanket to provide rapid relief of any hydrostatic pressure behind the blanket. Filter blankets are generally necessary as in the case of other types of rip rap.

<u>Sacked Concrete Rip Rap Design</u> - Concrete rip rap in bags generally consist of approximately 2.3 cubic foot of concrete in a burlap bag. This type of rip

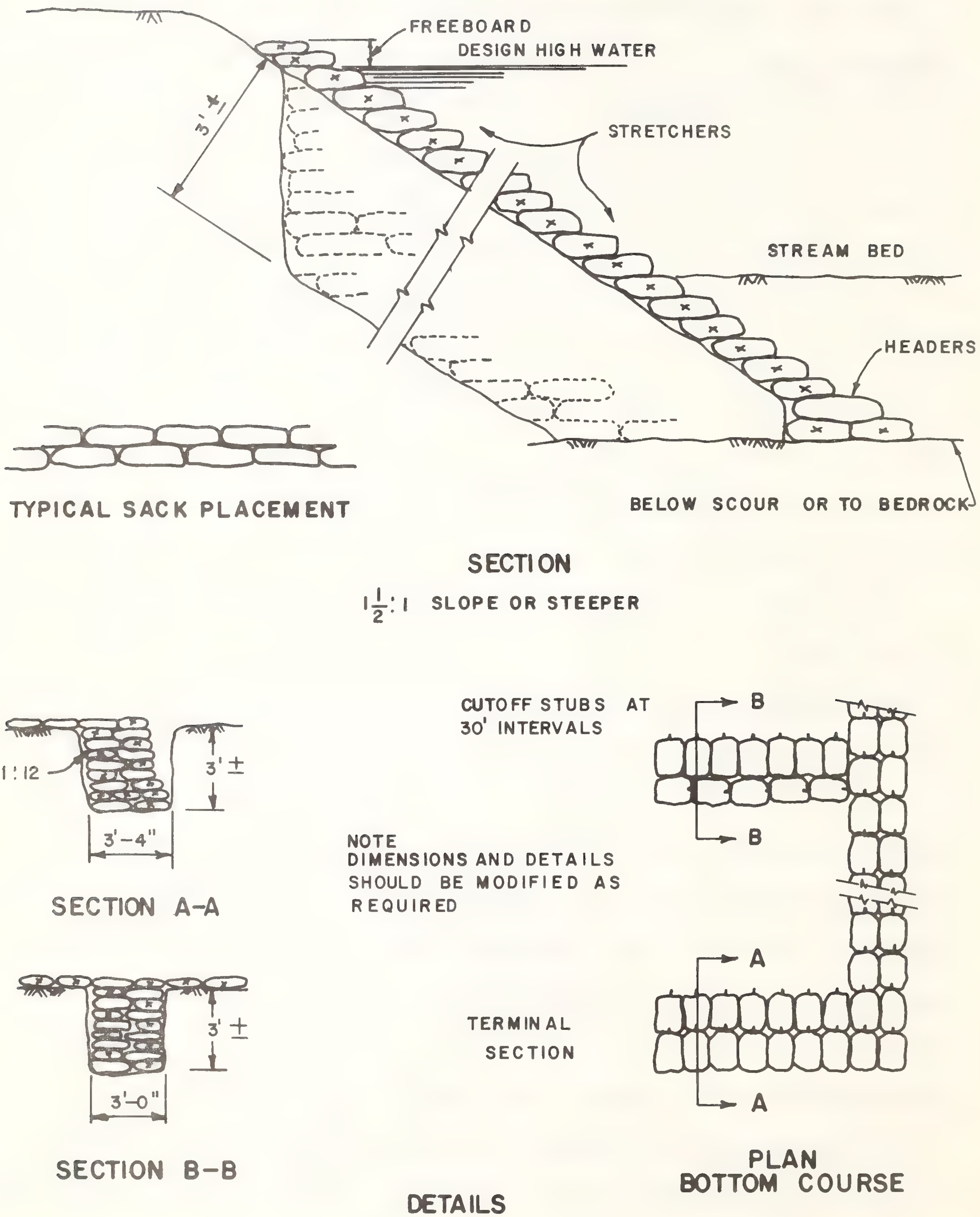


FIG. 4.59-TYPICAL SECTION AND DETAILS OF SACKED CONCRETE SLOPE PROTECTION

rap provides a heavy protection regardless of the requirements of the site. The rip rap has little flexibility, low tensile strength and is susceptible to damage from floating ice. It requires firm support from the protected bank and usually requires a filter blanket underneath the rip rap. Adequate protection of the terminals and toe is essential. The toe trench must end in firm support and extend below the depth of anticipated scour. Details of terminal protection and cutoff stubs are shown in Figure 4.59.

The bags make close contact with each other and some bond is secured between the bags by the cement mortar leaking through the porous bags. Flat slopes reduce the area of contact between the sacks and thus bond is less. Slopes of the protected embankment are generally 1 1/2:1. If the slopes are flat as 2:1, all sacks after the bottom row should be laid as headers (long way of sack in line with the slope) rather than as stretchers (long way at right angles to slope direction).

Concrete Slab Slope Protection - Concrete slabs, plain or reinforced, are cast in place on the prepared slope. The slab is generally 4 inches thick of Class DD concrete. Precast concrete slabs can be used in place of cast-in-place slabs.

Design of Filter Blankets - A filter blanket is usually needed beneath the rip rap cover to prevent the water from removing bank material through voids in the rip rap. Removal of bank material leaves cavities behind the rip rap cover and failure of the cover might result, particularly if the rip rap cover is rigid and cannot slump to continue contact with the supporting soil. Whether a filter blanket is needed will depend upon the gradation of the bank material and the openings or voids in the rip rap cover. For random rip rap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15 percent particle size (D_{15}) of the coarser layer to the 85 percent particle size (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent particle size of the coarser material to the 15 percent particle

size of the finer material should exceed 5 but be less than 40. This requirement can be stated thus:

$$\frac{D_{15}\text{ (of rip rap)}}{D_{85}\text{ (of bank)}} < 5 < \frac{D_{15}\text{ (of rip rap)}}{D_{15}\text{ (of bank)}} < 40$$

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter blanket material if more than one layer is used, and between the filter blanket and the stone cover. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of the fine material into the coarser material. Not more than 5 percent of the filter material should pass the No. 200 sieve.

The thickness of the filter blanket ranges from 6 inches to 15 inches for a single layer or from 4 inches to 8 inches for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, thickness of the blanket layers should approach the minimum. Thickness of individual layers should be increased above the minimum proportionately as a gradation curve of the material comprising the layer departs from a parallel pattern.

An example of a filter design for random rip rap follows. It is assumed that rip rap is to be used to protect a streambank with the gradation of the filter shown in Figure 4.60. The gradation curves of the rip rap and of the sand and gravel available for use are also shown on Figure 4.60.

Example 1

Material	Particle Size D_{15}	Particle Size D_{85}
Rip Rap	90 mm	308 mm
Streambank	0.006 mm	0.10 mm
Sand	0.14 mm	2.4 mm
Gravel	4.0 mm	50 mm

Is filter required?

$$\frac{D_{15}\text{ (rip rap)}}{D_{85}\text{ (Streambank)}} = \frac{90}{0.10} = 900 > 5 \qquad \text{Yes.}$$

Can a single layer of gravel be used?

$$\frac{D_{15}\text{ (rip rap)}}{D_{85}\text{ (gravel)}} = \frac{90}{50} = 1.8 < 5 \qquad \text{O.K.}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{85}\text{ (streambank)}} = \frac{4.0}{0.10} = 40 > 5 \qquad \text{No.}$$

Can a layer of sand and a layer of gravel be used?

1st Requirement

$$\frac{D_{15}\text{ (rip rap)}}{D_{85}\text{ (gravel)}} = \frac{90}{50} = 1.8 < 5 \qquad \text{O.K.}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{85}\text{ (sand)}} = \frac{4.0}{2.4} = 1.7 < 5 \qquad \text{O.K.}$$

$$\frac{D_{15}\text{ (sand)}}{D_{85}\text{ (streambank)}} = \frac{0.14}{0.10} = 1.4 < 5 \qquad \text{O.K.}$$

2nd Requirement

$$\frac{D_{15}\text{ (rip rap)}}{D_{15}\text{ (gravel)}} = \frac{90}{4.0} = 22 < 40 \qquad \text{O.K.}$$

$$\frac{D_{15}\text{ (gravel)}}{D_{15}\text{ (sand)}} = \frac{4.0}{0.14} = 29 < 40 \qquad \text{O.K.}$$

$$\frac{D_{15}\text{ (sand)}}{D_{15}\text{ (streambank}} = \frac{0.14}{0.006} = 23 < 40 \qquad \text{O.K.}$$

The gradation of the sand and the gravel is satisfactory; if adequate placing methods are to be used, two minimum thickness layers (4 or 5 inches) can be used, one of sand and one of gravel.

For rip rap other than dumped stone, the maximum size of openings in the cover is used as the criterion for filter design. Then:

$$\frac{D_{85}\text{ of the filter}}{\text{Maximum opening in cover}} = 2 \text{ or more}$$

When weep holes are used in a solid cover, an inverted filter should be used under the weep holes in addition to the filter blanket.

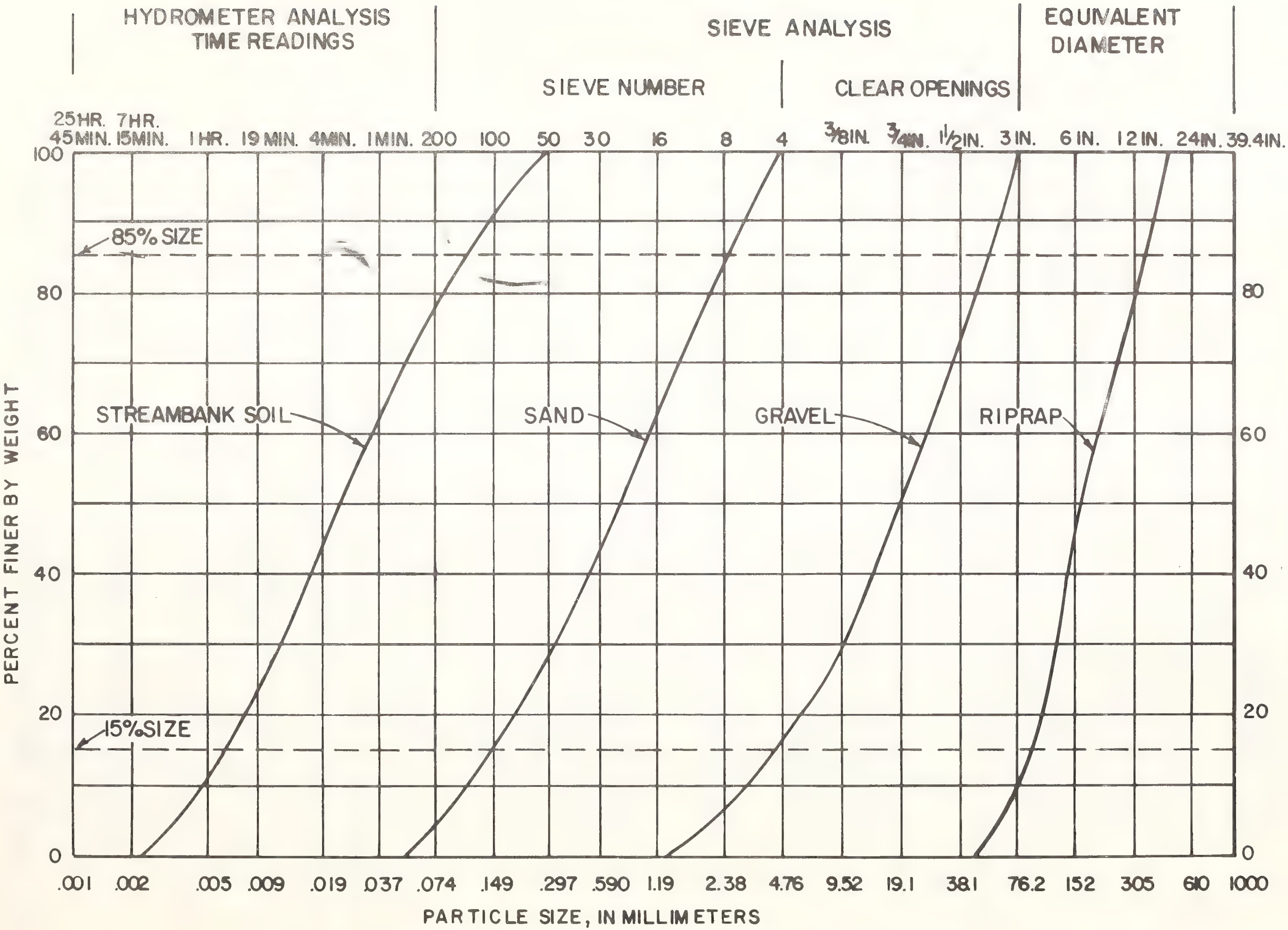


FIG. 4.60— GRADATION CURVES FOR FILTER DESIGN

4.82 DITCH LININGS

Introduction

One means of reducing erosion on the highway right-of-way during construction and operation is through the use of properly designed linings in drainage channels. Linings may be rigid, such as portland cement or asphaltic concrete, or flexible, such as grass or rock rip rap.

Rigid linings are expensive to construct and maintain, have an unnatural appearance, prevent or reduce natural infiltration, and contribute to high velocities and scour at the end of the lining unless roughness elements are added to slow the flow. Many rigid linings are destroyed due to flow under-cutting the lining, headcutting, or hydrostatic pressure behind the channel walls or floor. Flexible linings are generally cheaper to install, have self-healing qualities which reduce maintenance costs, permit infiltration and exfiltration, and have a natural appearance, especially after vegetation has been established.

When grass or other vegetation is chosen as the permanent channel lining, it may be established by sodding or seeding. Installation by sodding is expensive but has the advantage or providing immediate protection. Install-ation by seeding usually requires protection by one of a variety of temporary lining materials until the vegetation becomes established.

Considerable research has been performed in the past several years on temporary lining materials and rock rip rap channel lining design. The purpose of this section is to present that information in a useable form, coupled with information on vegetative linings. Rigid linings will be covered to a limited extent as related to the flexible lining materials. However, the hydraulic design of rigid linings is covered in detail in Section 4.6, Open - Channel Flow.

The majority of the material for this section was taken from Hydraulics Engineering Circular No. 15.

Lining Material Design - The basic design method presented in this section is based on the concept of maximum permissible depth of flow, coupled with the hydraulic resistance of the particular lining material. In all cases, the lining material defines the hydraulic resistance of the channel while providing its own peculiar degree of erosion protection.

Erosion Prevention - The d_{max} charts are used to define the maximum permissible depth of flow for a particular lining, based on channel slope, S_o, and the erodibility of the underlying soil. Maximum permissible depth is based on the tractive force theory of channel lining design. For wide channels of any shape, and for a given channel slope, depth, and lining, the vertical velocity distribution in the central and deepest section, where wall effects are negligible, should be identical. Also, the first scour occurs at the deepest portion of the channel, since the wall or bottom shear stress is greatest in that portion.

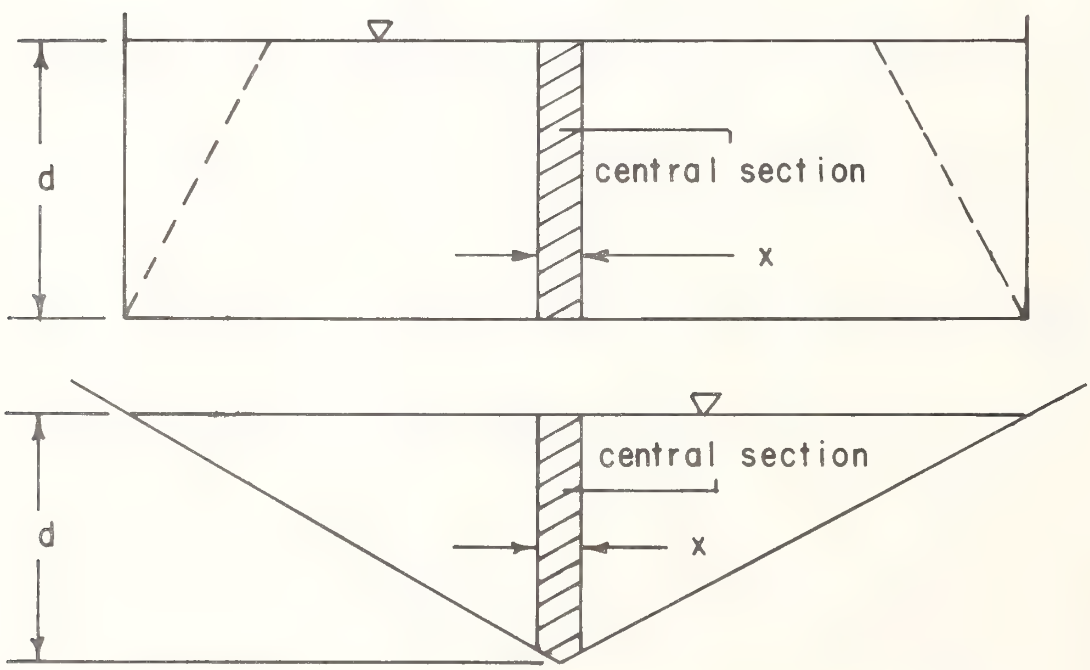


Fig.4.61 Schematic diagram of channels of different shapes

If the depth of flow, channel slope, lining, and soil are the same in both channels shown in Figure 4.61, then the flow rate and the mean channel velocity for the two channels will be different, but in the central section of both channels, represented by X, the velocity distribution and bottom shear stress will be nearly identical. Therefore, in both channels, there exists a limiting depth of flow above which scour will occur, and this depth, d_{max}, is the same for all wide channels of the same longitudinal slope, lining, and underlying soil.

From Chart 4.128, notice that as the efficiency of the lining increases, the influence of the underlying soil on d_{max} decreases. For instance, the unprotected soil shows a greater range of d_{max} than the channels lined with fiber glass roving and asphalt.

Definition of the erodibility of cohesive soils is an area which has thus far eluded quantitative definition. It is suggested that the erodibility of specific soils be based on the designer's experience rather than any quantitative analysis based, for example, on plasticity index. Based on the work at Mississippi State University, which covered ten soils of different characteristics, soils with a gravel, sand and clay mixture are erosion resistant; fine-grained sands or silts are erodible; and plastic and semi-plastic soils are in the intermediate range. The soil erodibility factor for the Universal Soil-Loss Equation, developed by the Agricultural Research Service, could also be used as a guide to soil erodibility. If the designer has no knowledge of the erodibility of the soil at a particular channel site, a reasonable estimate of d_{max} may be obtained by interpolating half-way between the "erosion resistant" and "erodible" lines of the maximum permissible depth charts.

Hydraulic Resistance - The flow velocity charts were developed to define the relationship between the hydraulic radius of the channel, longitudinal slope of the channel (S_o), and mean channel velocity (V) for a given channel lining.

For some linings, such as rock rip rap and fiber glass roving tacked with asphalt, the Manning equation may be used. For rock rip rap, the Manning n value varies with mean stone size, as follows:

$$n = 0.0395\ D_{50}^{1/6}$$

For fiber glass roving tacked with asphalt, it was found that the Manning n value was approximately a constant:

	Smooth rolled channels	Channels with clods and tracks
Single Layer	0.030	0.035
Double Layer	0.020	0.025

The higher values of n were used in the development of Charts 4.130 and 4.141, assuming that most channels will be rather rough after seeding and mulching.

For the other channel linings, Manning n values were found to vary with slope and hydraulic radius, and thus empirical curves were developed to represent the data. Equations are given for the unlined channel and the other temporary channel linings. For the vegetative linings of various retardances, curves were taken directly from the SCS Handbook of Channel Design. Retardance is the hydraulic resistance relationship for a certain group of grasses of given lengths as defined by the SCS. Retardance A refers to grasses of high hydraulic resistance, while Retardance E refers to grasses of very low hydraulic resistance.

Design Flow Rate - The design flow rate for roadside and median drainage channel linings are usually estimated for a 5 or 10 year recurrence interval. If a vegetative lining is feasible, and temporary lining is to be used for the establishment period, a lower recurrence interval might be considered for the design of the artificial lining. This is because the risk of damage to the temporary lining during this relatively short time is quite low, and if the lining is damaged, repairs are quite inexpensive.

Selection of the design flow rates should be based on some type of a subjective risk analysis. If the available channel linings are found to be inadequate for the selected design flow rate, it may be feasible to use inlets to intercept some flow and reduce the flow rate to a manageable level.

Channel Side Slopes - For purposes of safety, construction, maintenance, and erosion resistance, it is suggested that the channel side slopes be kept as flat as possible. Flatter slopes may be necessary for safety or other reasons.

Analysis of rip rap design methods show that if a rip rap lining is used with 3:1 or flatter side slopes, there is no need to check the sides for scour. With steeper side slopes, the combination of velocity against the stone and gravitational effects may dislodge the stone on the sides before the bottom is disturbed. Methods of checking steeper rip rap lined sides will be presented later.

The SCS recommends 3:1 to 4:1 maximum side slopes on vegetative lined channels for ease of construction, mowing, and crossing the channel with equipment.

Outline of Design Procedure - The design procedures presented in this section consists of the following steps:

1. Perform hydrologic computations:
2. Select design flows for permanent lining material and for temporary linings based on subjective risk analyses.
3. Define soil erodibility.
4. Define channel shape, slope, and maximum top width.
5. Select least costly permanent lining material available.
6. Determine d_{max} for the selected lining, slope, and soil erodibility. (Maximum permissible depth charts).

7. Determine hydraulic radius (R) and area (A) for the selected channel geometry and d_{max}. (Chart 4.150 or other source).

8. Determine velocity from R and slope (S_o). (Flow velocity charts).

9. Allowable Q = AV.

10. If Q does not satisfy the design Q, select another channel size and return to step 4 or select another lining material and return to step 5. Also, consider the feasibility of additional inlets to reduce Q.

11. If a grass lining is the choice from the above computations, and it is desired to use a temporary lining material for channel protection during the period of grass establishment, perform steps 6, 7, 8, and 9 using the channel bottom width and side slopes from above with the selected temporary lining material and flow rate. The most stable temporary lining material is a double layer of fiber glass tacked with asphalt. However, this lining may retard vegetation germination and growth.

A computation sheet, shown in Figure 4.62 has been developed to facilitate the above design procedure using the charts of this section.

Flow in Bends - Flow around a bend in an open channel creates secondary currents which impose higher shear stresses on the channel sides and bottom. The location of the maximum shear varies depending on the position in the bend.

Chart 4.149 has been prepared to adjust the D_{50} of rock rip rap lining for the higher shear stresses in a bend, depending on the surface width of the channel (B_S) and the mean radius of the bend (R_o). To use the chart, determine K_3 based on the ratio B_S/R_o. Then, multiply the D_{50}, determined for the straight channel reaches, by K_3 to obtain a larger stone size for use in the bend. For instance, assume that a channel with a top width of 12 feet requires a stone size of D_{50} = 0.5 feet in the straight reaches. Assume that

the channel has a bend with a 24 foot radius. The $B_S/R_O = \frac{12}{24} = 0.5$ from Chart 4.149, for $B_S/R_O = 0.5$, $K_3 = 1.18$.

$$(D_{50})\ \text{Bend} = 1.18\ (D_{50})\ \text{Straight}$$
$$= (1.18)(0.5) = 0.59'.$$

Therefore, the stone in the bend area should have a D_{50} of 0.59 feet. Since it is not possible to predict the location of the maximum shear, the entire channel cross section must be protected with the same stone.

Chart 4.149 also indicates the B_S/R_O values requiring additional protection with other lining materials. It appears that as long as B_S/R_O is less than 0.4, no additional protection is necessary. When B_S/R_O is greater than 0.4, the lining in the bend area should be reinforced. For instance, with a single layer of fiber glass roving and asphalt in the straight channel, a double layer could be used in the bend.

Rigid Linings - For rigid channel linings, such as concrete, there is no maximum permissible depth for the flow velocities normally encountered in highway drainage work. Thus, the maximum depth is based only on the freeboard requirement for the channel. The Manning equation may be solved by trial and error, or the charts of Section 4.6 Open Channel Flow can be used for the design of rigid channel linings.

Vegetative Linings - The procedure for the design of vegetative linings for channels was developed by the SCS and summarized in the publication, Handbook of Channel Design for Soil and Water Conservation. The design of grass lined channels is more difficult because the roughness coefficient, n, varies with the type and height of grass and with the velocity and depth of flow.

The effect of the height and type of grass is defined by the retardance which is given in Table 4.38. The maximum permissible depth of flow is given in Chart 4.122 for various retardances and grass types. The flow velocities

Fig.4.62

Drainage Channel Lining Design

DATE: ______

PROJECT: ______ DESIGNER: ______

STATION ______ TO STATION ______ CHECKED BY ______

DRAINAGE AREA = ______ ACRES DATE ______

HYDROLOGIC COMPUTATIONS:

DESIGN FLOW: Q ____ = ______ cfs

DESIGN FLOW FOR TEMPORARY LINING: Q ____ = ______ cfs

SOIL ERODIBILITY: ______

CHANNEL DESCRIPTION: MAX. TOPWIDTH = ______ ft.

S_o = ______

AVAILABLE LININGS:

LINING	d_{max}	B	$\frac{d_{max}}{B}$	$\frac{A}{Bd}$	A	$\frac{R}{d}$	R	V	Q=AV	T	REMARKS

are given in Charts 4.123 - 4.127.

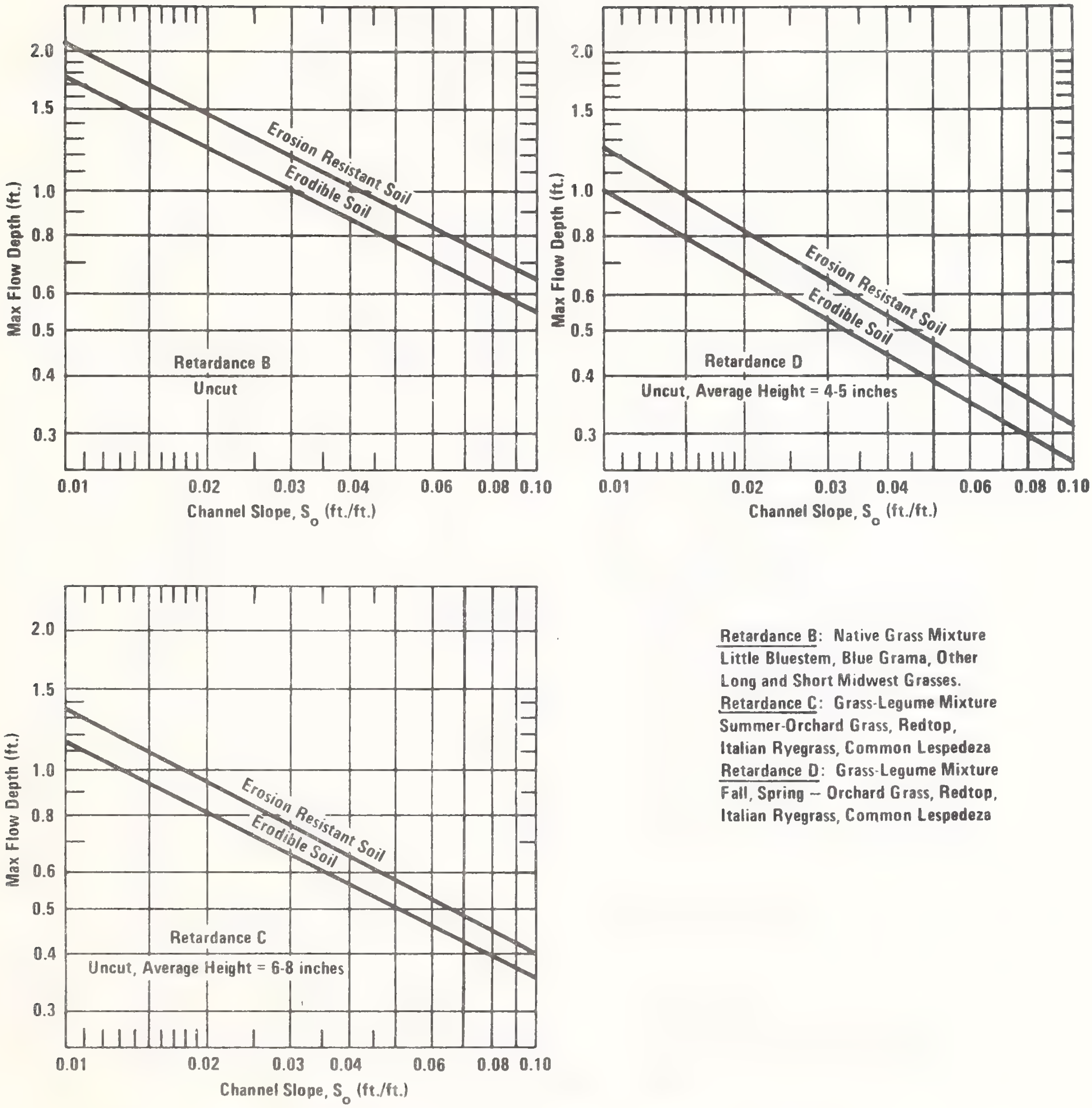
Additional nomgraphs are given in Section 4.63 for the solution of open channel flow in grass lined channels.

TABLE 4.38 Classification of vegetel covers as to degree of retardance

Note: Covers classified have been tested in experimental channels. Covers were green and generally uniform.

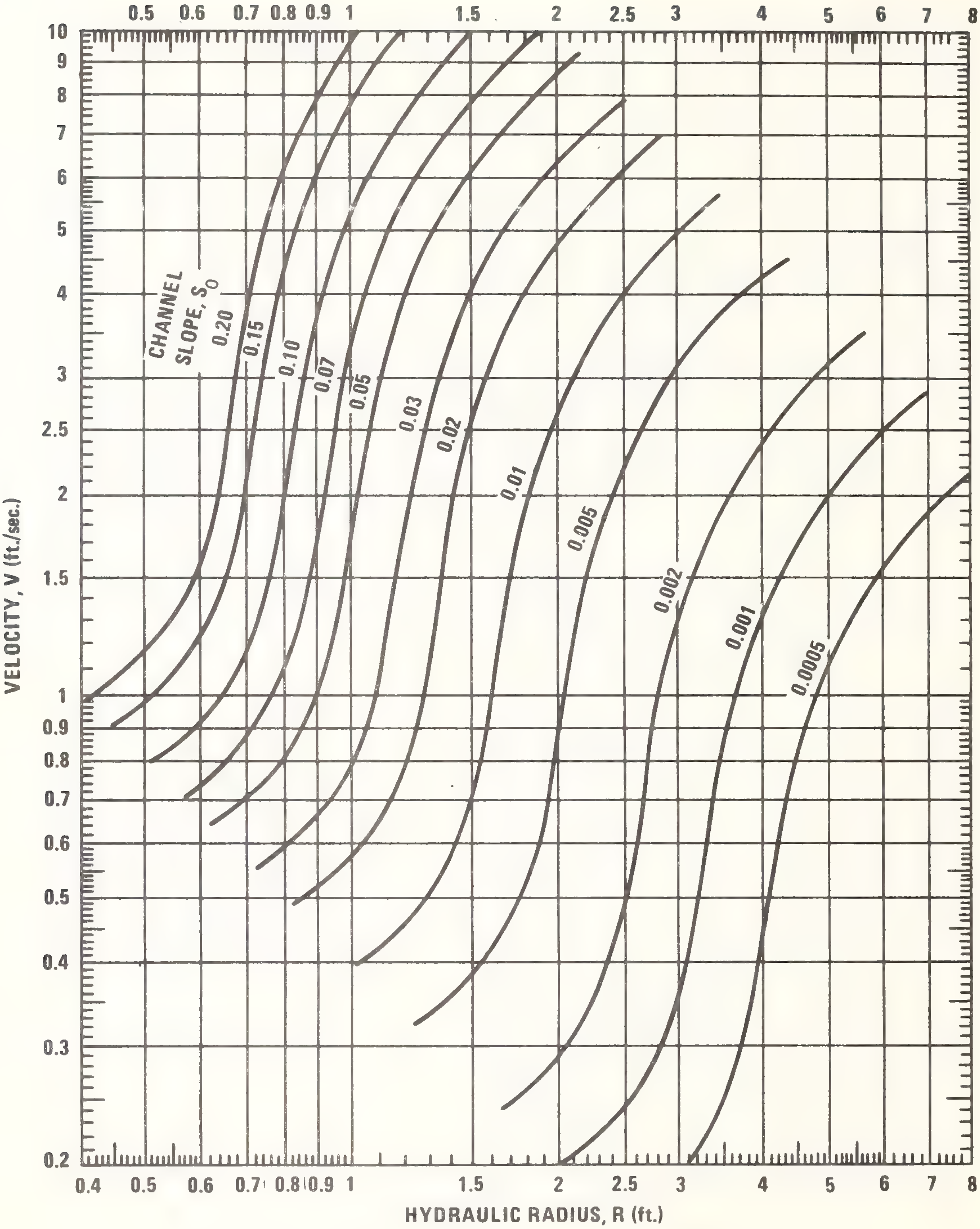
Retardance	Cover	Condition
A	Weeping lovegrass...........	Excellent stand, tall, (average 30")
	Yellow bluestem Ischaemum....	Excellent stand, tall, (average 36")
B	Kudzu......................	Very dense growth, uncut
	Bermudagrass................	Good stand, tall (average 12")
	Native grass mixture (little bluestem, blue grama, and other long and short mid-west grasses).............	Good stand, unmowed
	Weeping lovegrass...........	Good stand, tall, (average 24")
	Lespedeza sericea...........	Good stand, not woody, tall (average 19")
	Alfalfa.....................	Good stand, uncut, (average 11")
	Weeping lovegrass...........	Good stand, mowed, (average 13")
	Kudzu......................	Dense growth, uncut
	Blue grama..................	Good stand, uncut, (average 13")
C	Crabgrass...................	Fair stand, uncut (10 to 48")
	Bermudagrass................	Good stand, mowed (average 6")
	Common lespedeze............	Good stand, uncut (average 11")
	Grass-legume mixture--summer (orchard grass, redtop, Italian ryegrass, and common lespedeza).........	Good stand, uncut (6 to 8")
	Centipedegrass..............	Very dense cover (average 6")
	Kentucky bluegrass..........	Good stand, headed (6 to 12")
D	Bermudagrass................	Good stand, cut to 2.5" height
	Common lespedeza............	Excellent stand, uncut (average 4.5")
	Buffalograss................	Good Stand, uncut (3 to 6")
	Grass-legume mixture--fall, spring (orchard grass, red-top, Italian ryegrass, and common lespedeza).........	Good stand, uncut (4 to 5")
	Lespedeza sericea...........	After cutting to 2" height Very good stand before cutting
E	Bermudagrass................	Good Stand, cut to 1.5" height
	Bermudagrass................	Burned stubble

From SCS "Handbook of Channel Design for Soil and Water Conservation"



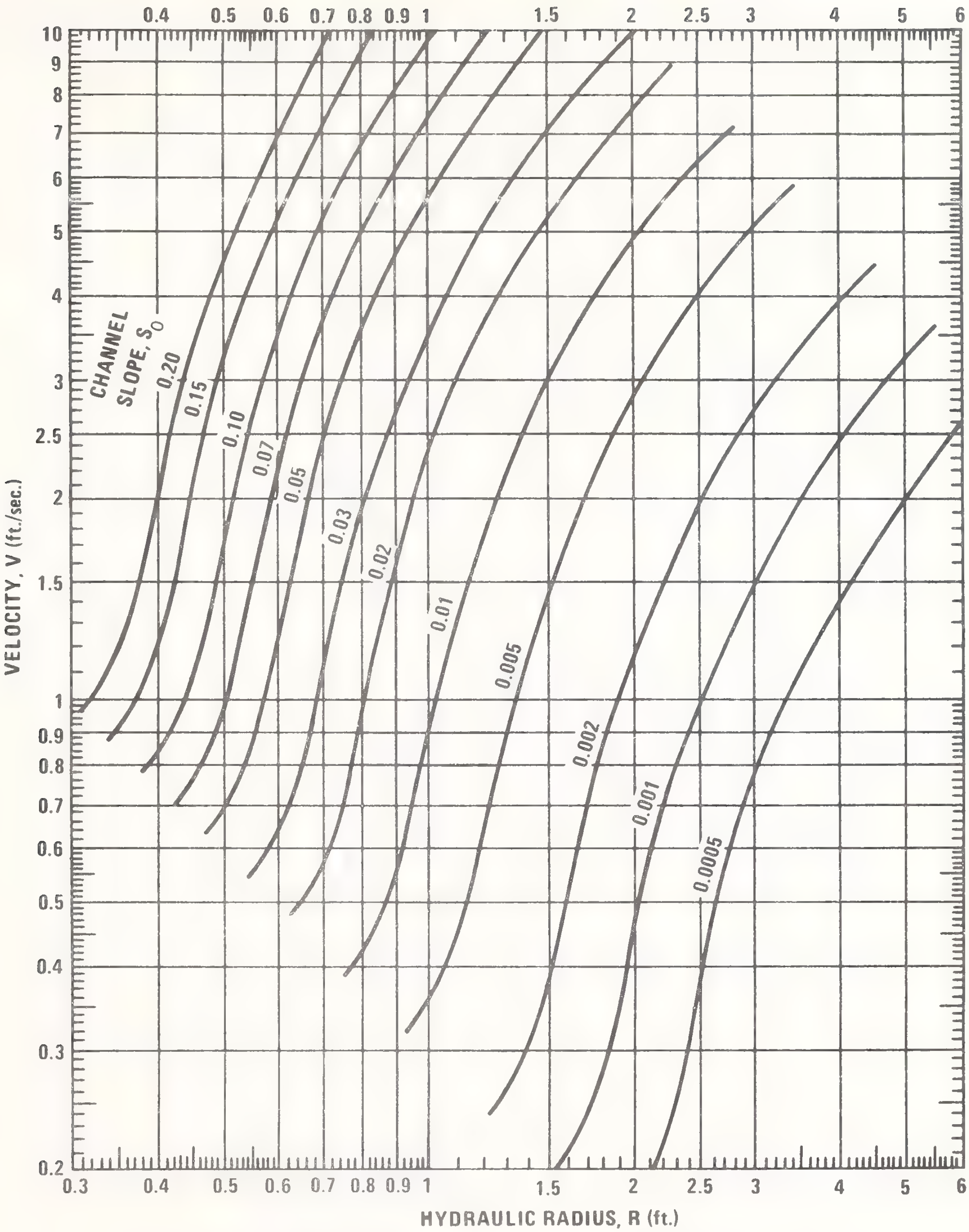
Retardance B: Native Grass Mixture
Little Bluestem, Blue Grama, Other
Long and Short Midwest Grasses.
Retardance C: Grass-Legume Mixture
Summer-Orchard Grass, Redtop,
Italian Ryegrass, Common Lespedeza
Retardance D: Grass-Legume Mixture
Fall, Spring -- Orchard Grass, Redtop,
Italian Ryegrass, Common Lespedeza

Maximum Permissible Depth of Flow (d_{max}) for Channels Lined with Grass Mixtures. Good Stand, Uncut



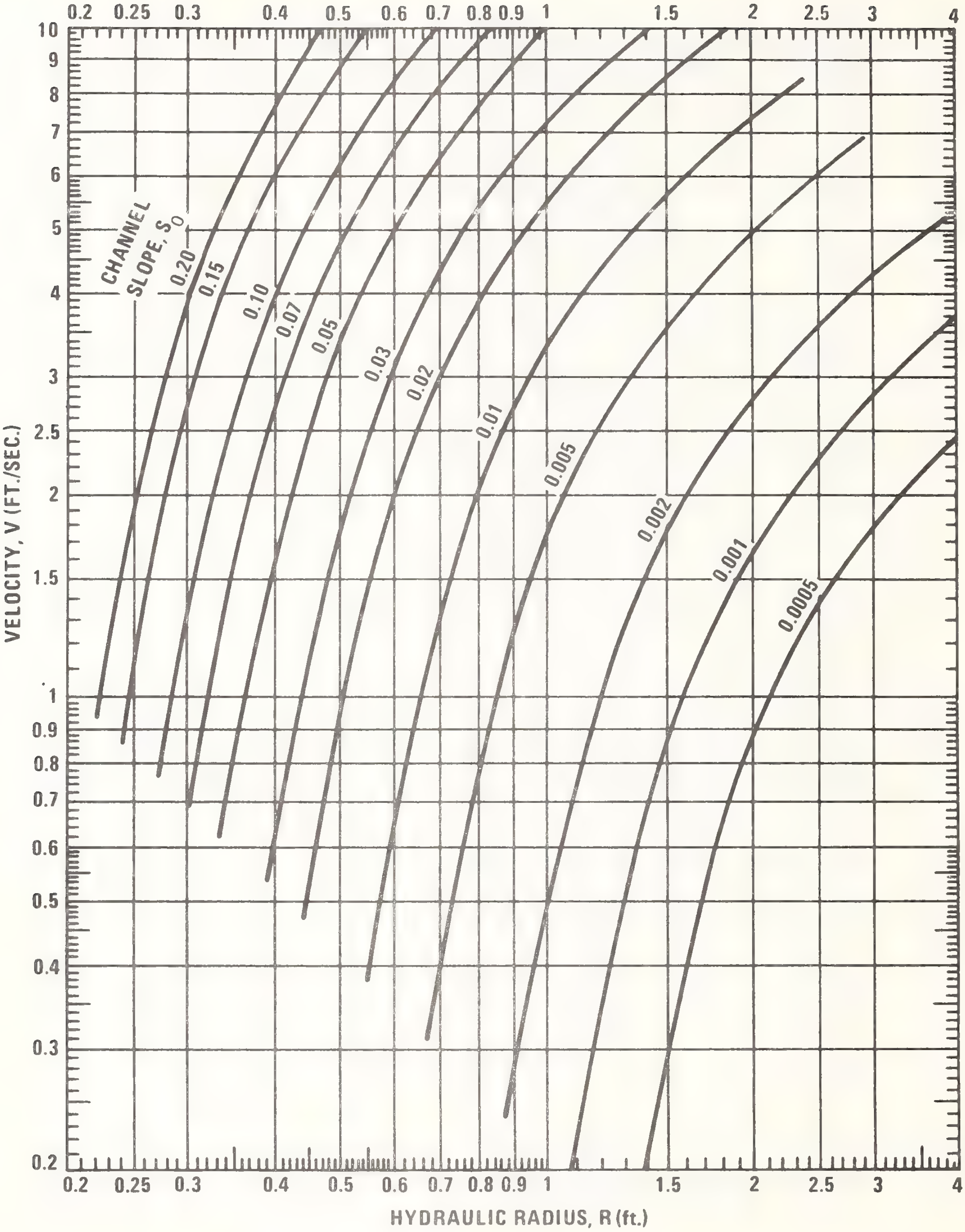
Flow Velocity for Channels Lined with Vegetation of Retardance A

From "Handbook of Channel Design for Soil and Water Conservation," SCS-TP-61, Revised 1954.



Flow Velocity for Channels Lined with Vegetation of Retardance B

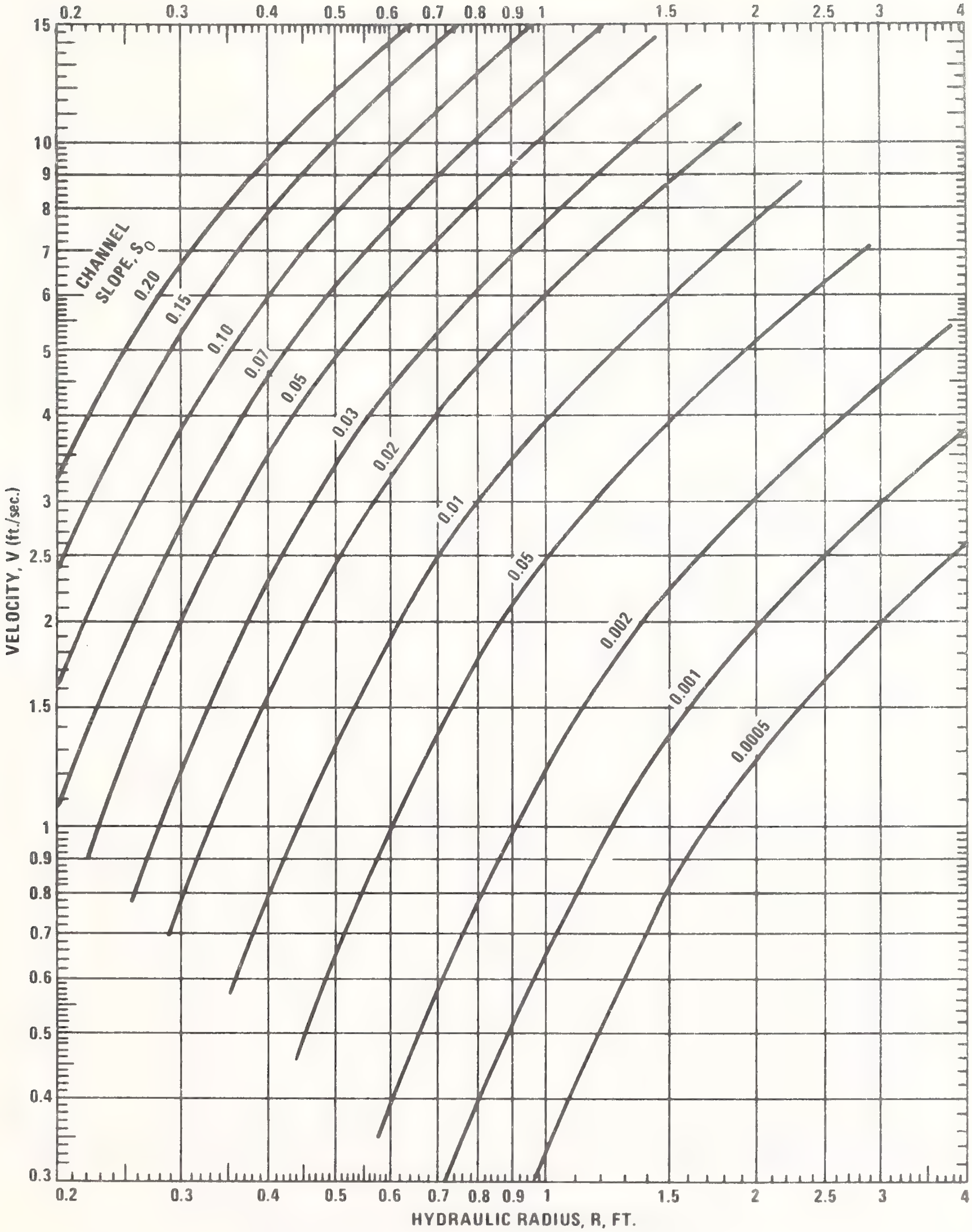
From "Handbook of Channel Design for Soil and Water Conservation," SCS-TP-61, Revised 1954.



Flow Velocity for Channels Lined with Vegetation of Retardance C

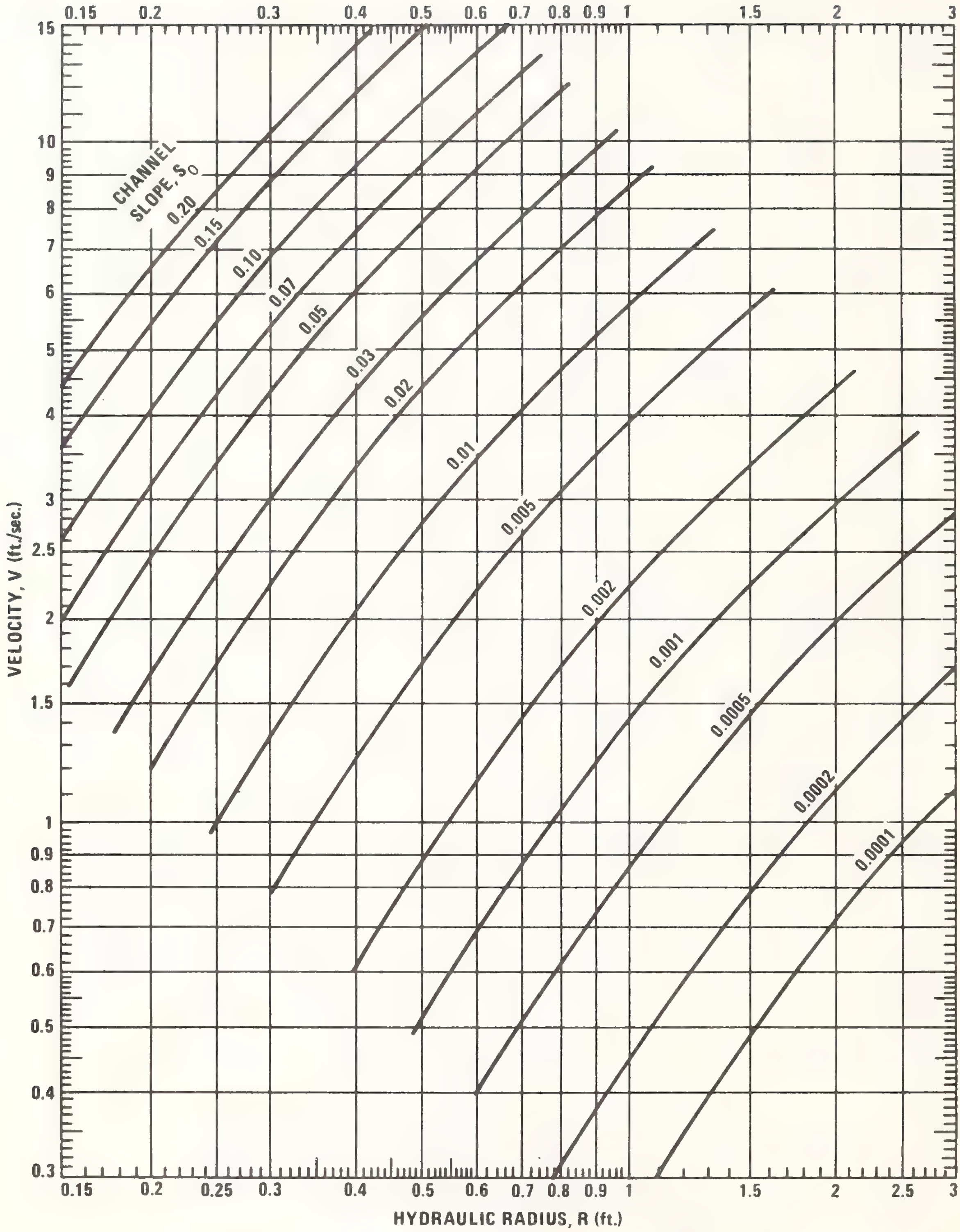
From "Handbook of Channel Design for Soil and Water Conservation," SCS-TP-61, Revised 1954.

Chart 4.126



Flow Velocity for Channels Lined with Vegetation of Retardance D

From "Handbook of Channel Design for Soil and Water Conservation," SCS-TP-61, Revised 1954.



Flow Velocity for Channels Lined with Vegetation of Retardance E

From "Handbook of Channel Design for Soil and Water Conservation," SCS-TP-61, Revised 1954.

Temporary Linings - The research on temporary linings was done at Mississippi State University in 1968 for the Mississippi State Highway Department under the Highway Planning and Research Program in cooperation with the Federal Highway Administration.

The following brief descriptions of temporary lining materials are not meant to be complete specifications. However, they should be adequate for the designer to determine whether the materials for which the design curves have been prepared are the same as those available to him.

1. Fiber Glass Roving
    a. Single layer - one layer of blown fiber glass fibers applied at a minimum rate of 0.25 pounds per square yard tacked with asphalt emulsion or asphalt cement at a minimum rate of 0.25 gallons per square yard.
    b. Double layer - two alternating layers of fiber glass and asphalt, each layer consisting of fiber glass roving at 0.25 pounds per square yard and asplalt at 0.25 gallons per square yard.
2. Jute Mesh
    a. Jute yarn which varies from 1/8- to 1/4- inch in diameter woven into a net which weighs approximately 0.80 pounds per square yard.
    b. Openings about 3/8- inch by 3/4- inch.
    c. Steel pins or staples spaced not more than 3 feet apart in 3 rows alternately spaced in the center. At the overlapping edges of parallel strips, staples shall be spaced at 2 feet or less. At all anchor slots, junction slots, and check slots, spacing shall be 6 inches or less. See Figure 4.63.
3. Excelsior Mat
    a. 0.8 pounds per square yard of excelsior (dried, shredded wood) covered with a fine paper net covering.

b. Paper net, reinforced along edges, with a spacing of approximately 1/2 inch by 2 inches.

c. Steel pins or staples placed at the rate of 5 staples per 6 linear feet of mat, placed two along each side and one in the middle. At the start of each roll use 4 or 5 staples spaced approximately one foot apart. Where more than one mat is required, they are butt-joined and securely stapled.

4. Straw with Erosionet

   a. Common oat straw applied at a rate of 3 tons per acre (1.25 pounds per square yard).

   b. Covered with Erosionet-315 (see description following).

   c. Place 3 staples on the end folds, one on each edge and one in the center. On the outside edges place staples at intervals of 5 to 8 feet. In the center place staples at intervals of 10 to 12 feet.

5. Erosionet-315

   a. Paper yarn approximately 0.05 inch in diameter woven into a net with openings approximately 7/8 inch by 1/2 inch.

   b. Weight about 0.20 pounds per square yard.

   c. Pinned in the same manner as jute mesh.

6. 1/2 inch Fiber Glass Mat

   a. A fine glass fiber mat similar to air filter material.

   b. Weight 0.35 pounds per square yard.

   c. Stapled in the same manner as Excelsior Mat.

7. 3/8 Inch Fiber Glass Mat

   a. Same material as No. 6.

   b. Weight 0.11 pounds per square yard.

   c. Stapled in the same manner as Excelsior Mat.

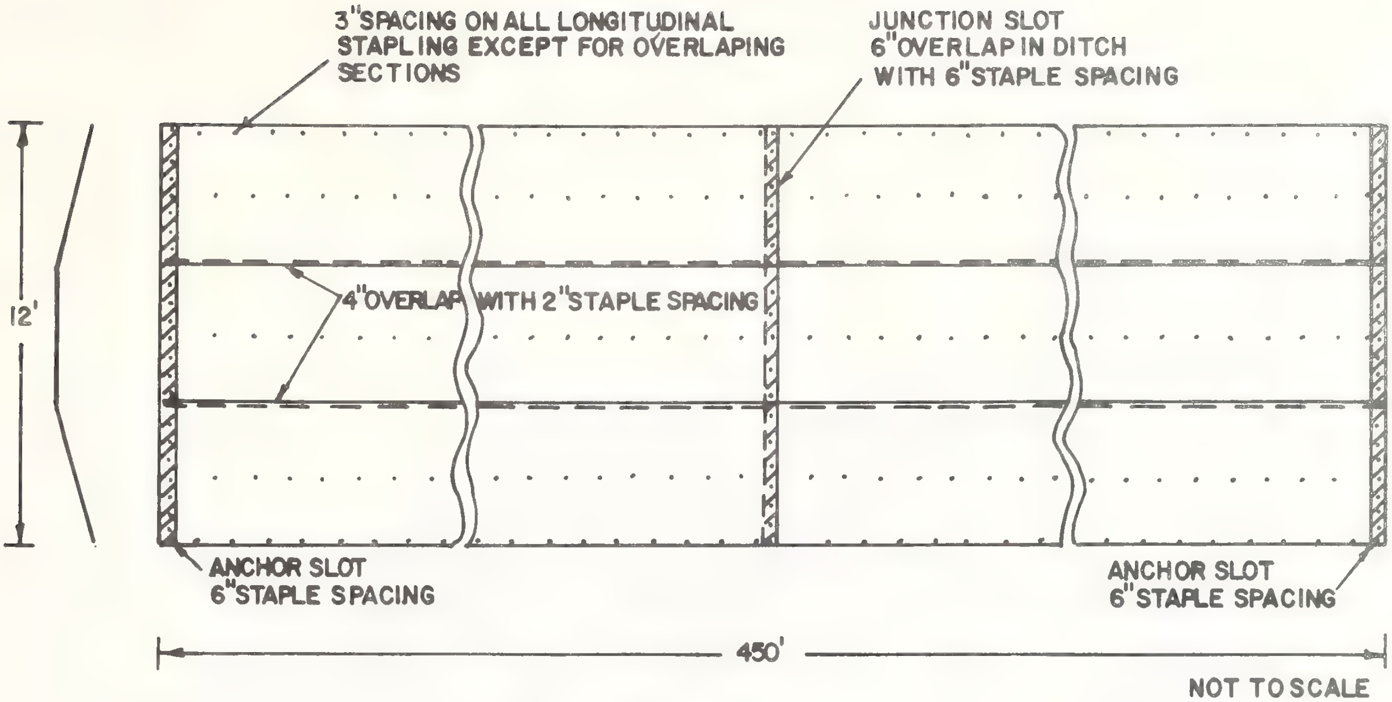
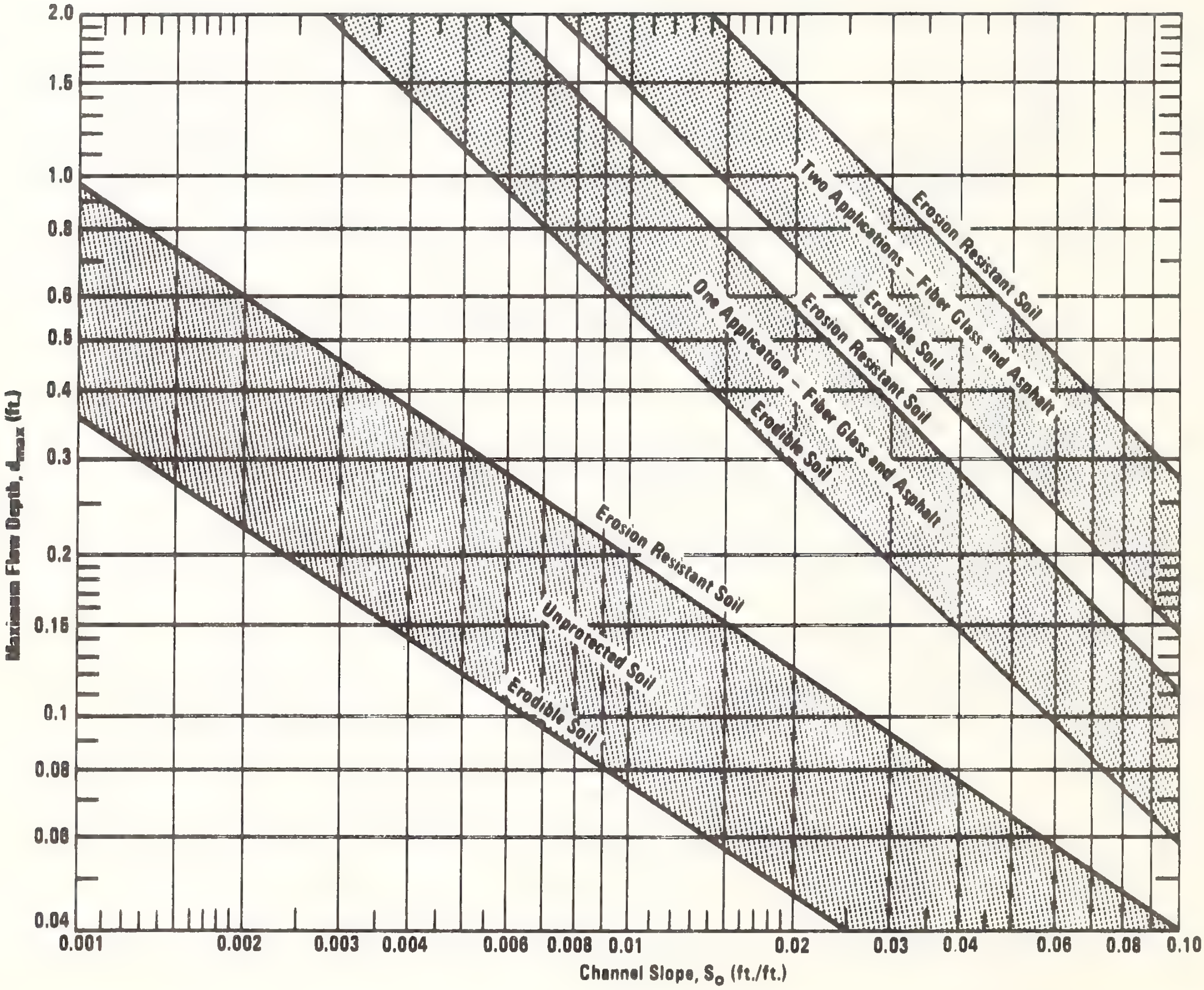


Fig. 4.63 STAPLING CONFIGURATION FOR JUTE MESH

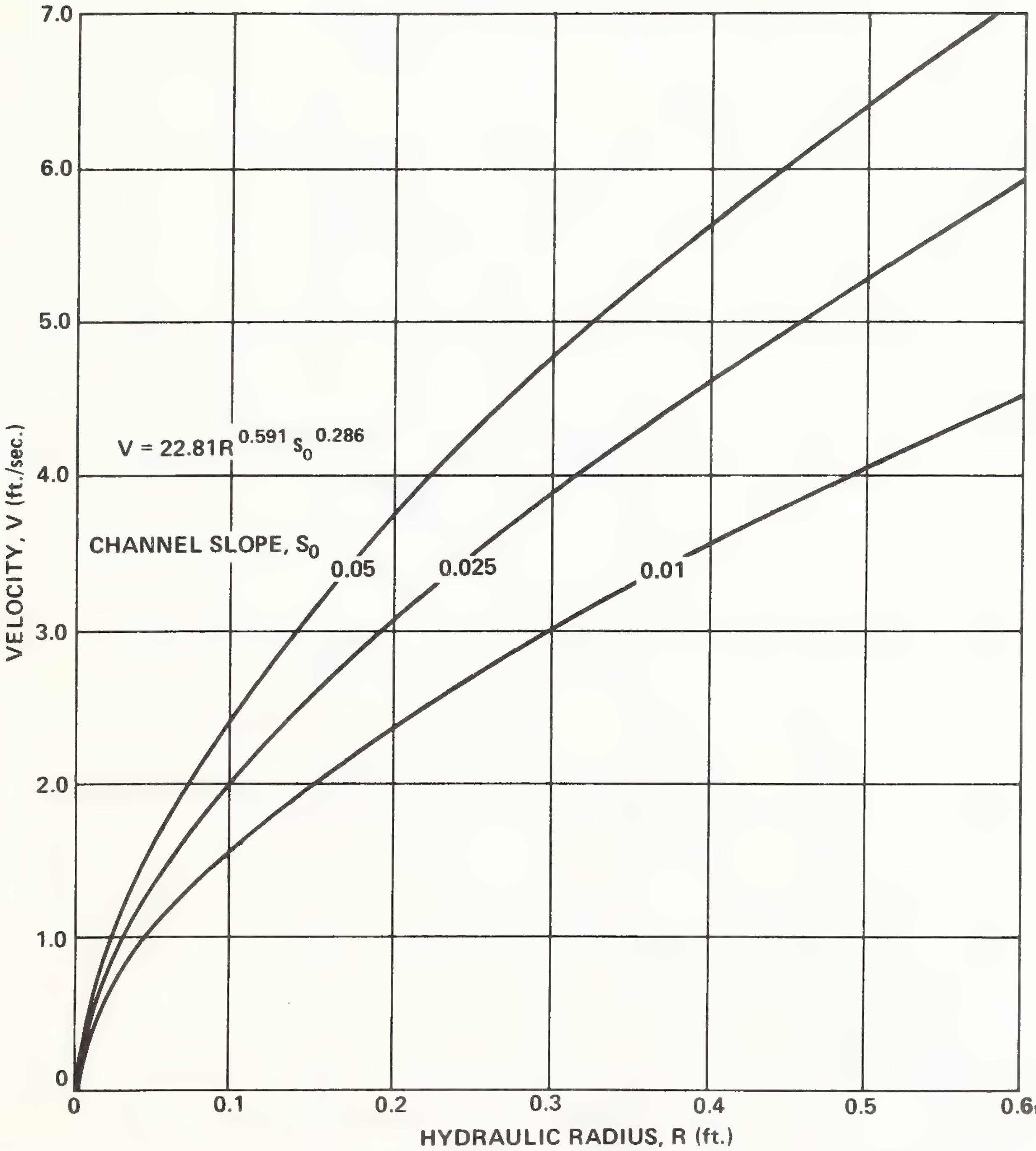
The maximum permissible depth of flow and the flow velocity for the various temporary linings are determined by using the corresponding charts of Chart 4.128 to Chart 4.143.



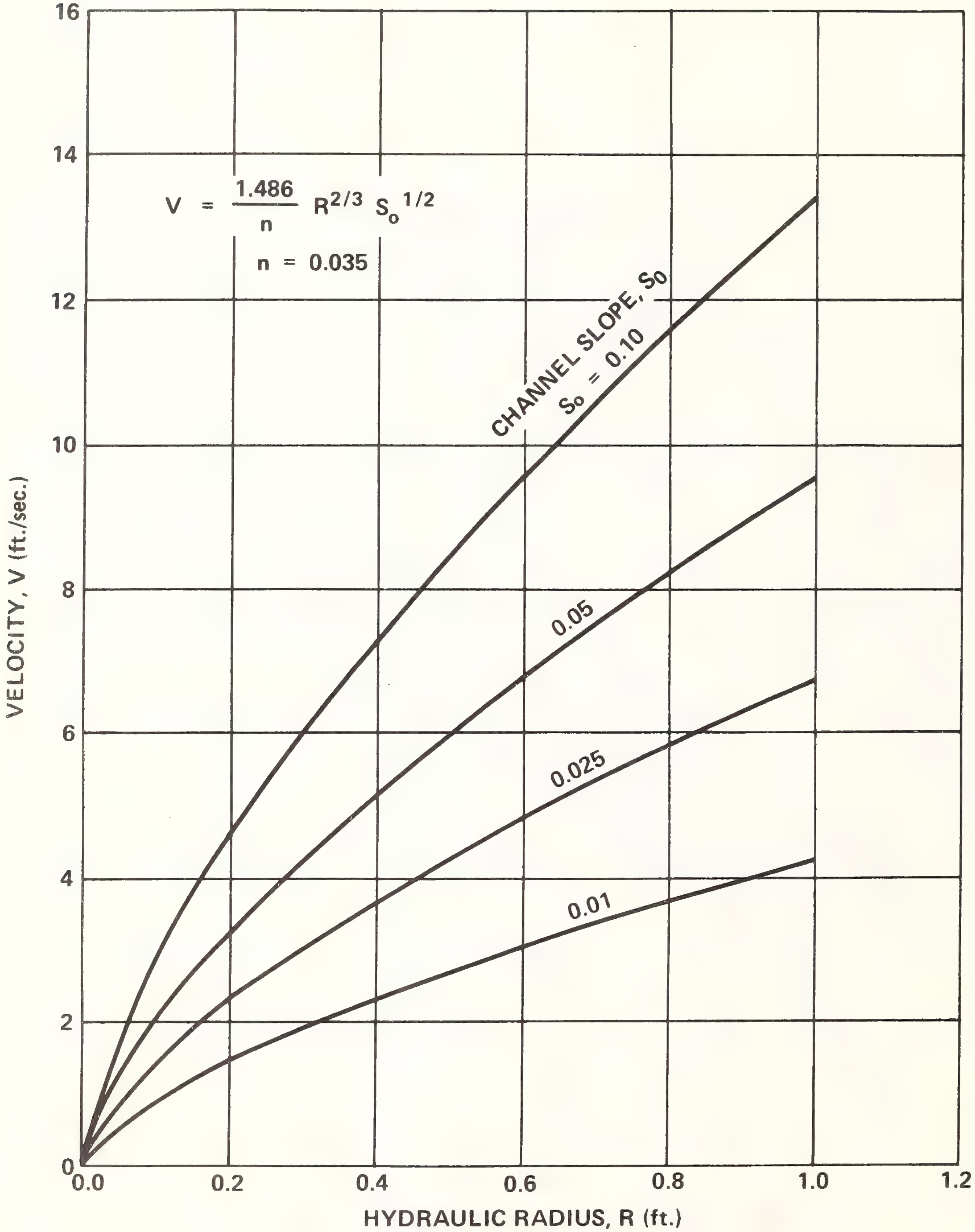
Maximum Permissible Depth of Flow (d_{max}) for Unlined Channels and Channels Lined with Fiber Glass Roving (Single and Double Layer)

Note: Fiber Glass Roving should be applied at least two feet beyond the anticipated high water during the vegetation establishment period

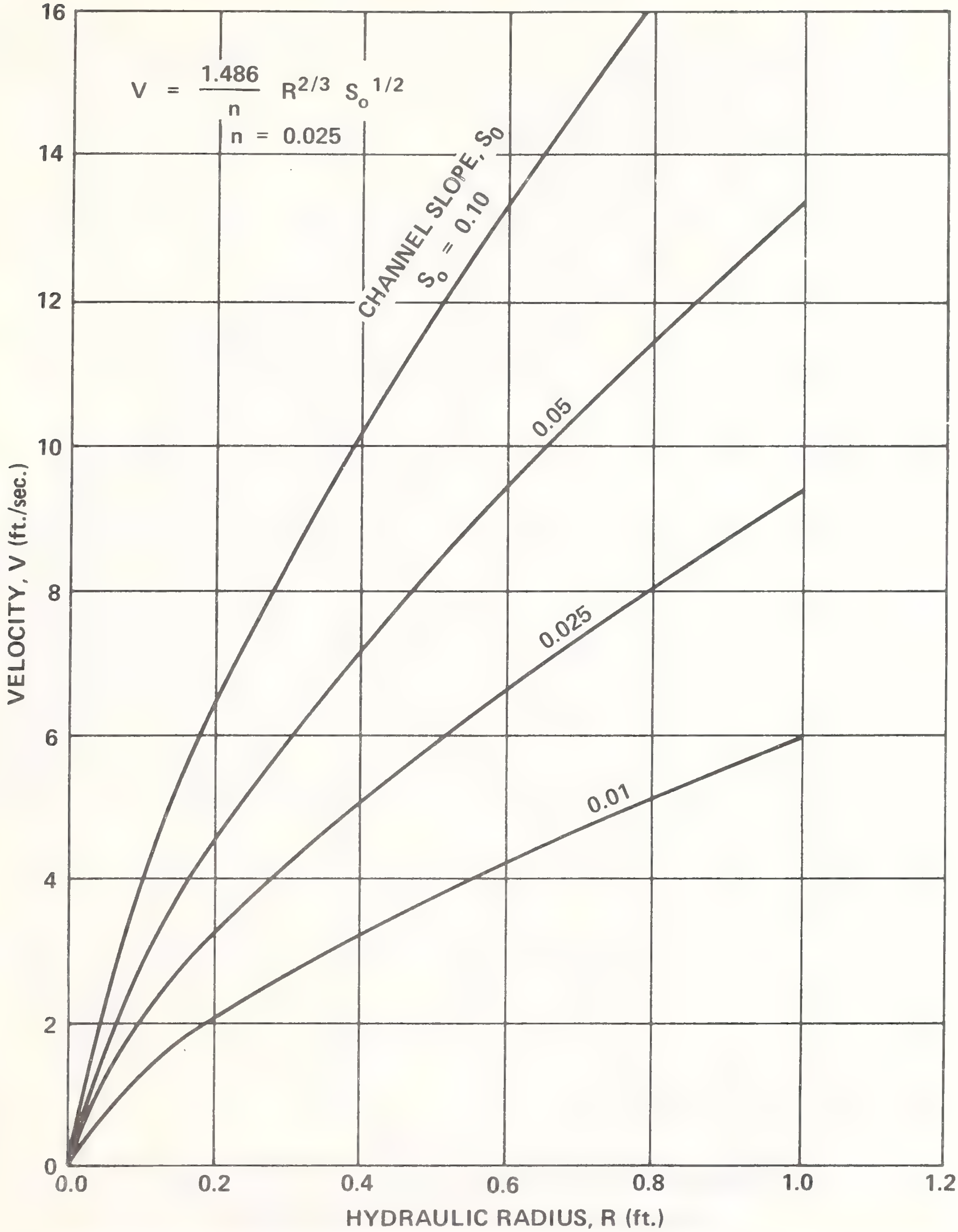
Chart 4.129



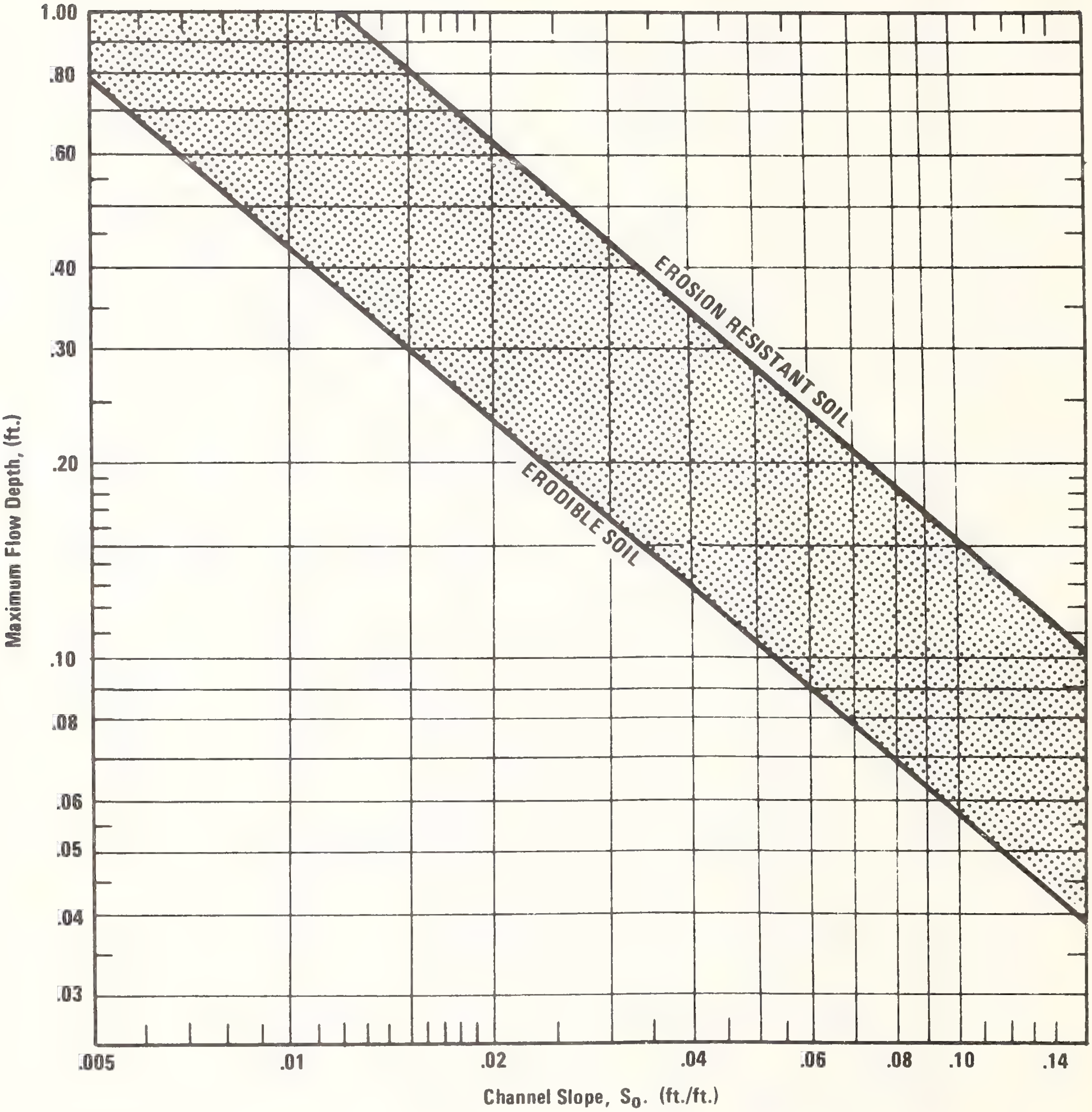
Flow Velocity for Unlined Channels (Bare Soil)



Flow Velocity for Channels Lined with Fiber Glass Roving Tacked with Asphalt, Single Layer

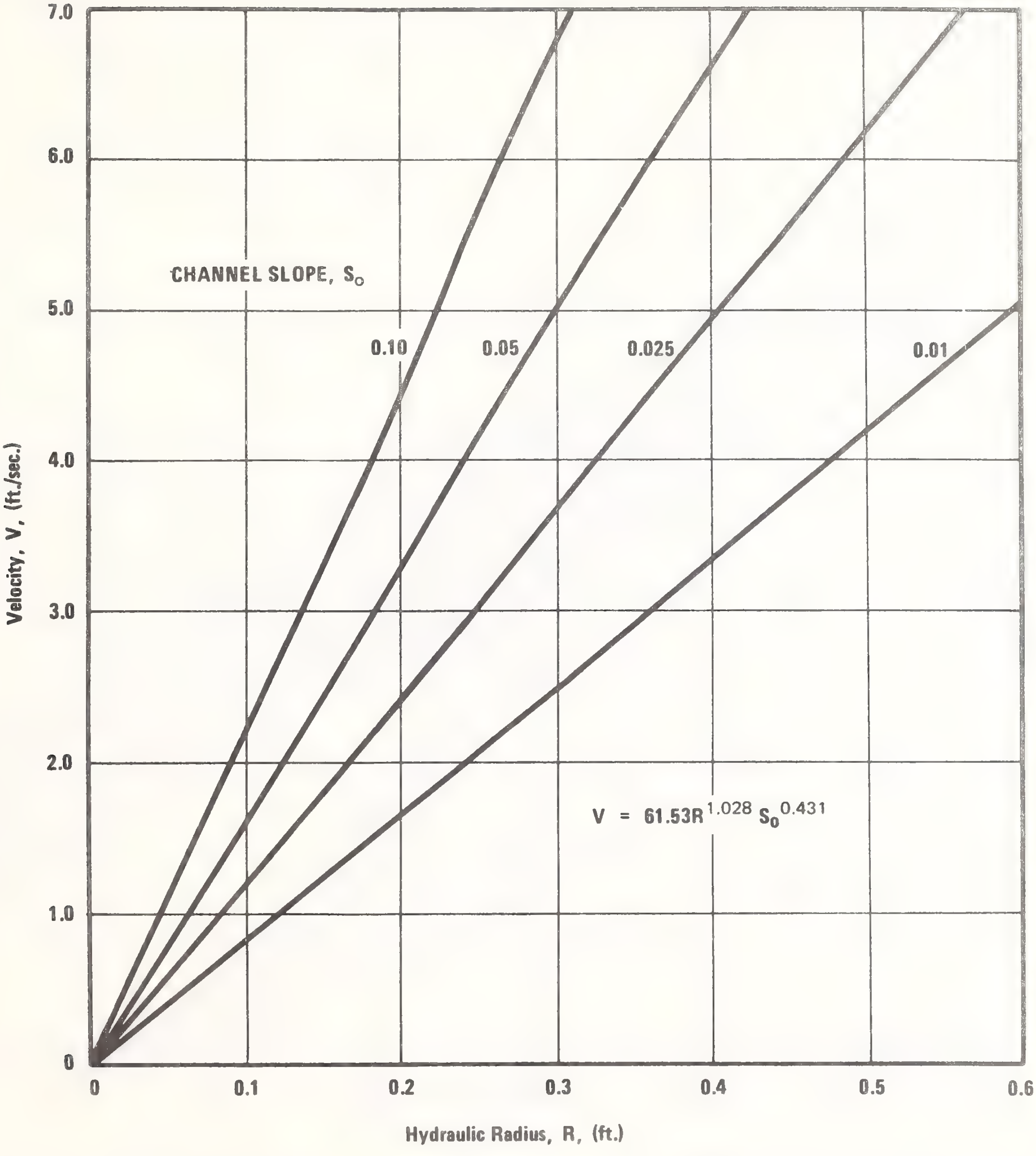


Flow Velocity for Channels Lined with Fiber Glass Roving Tacked with Asphalt, Double Layer



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

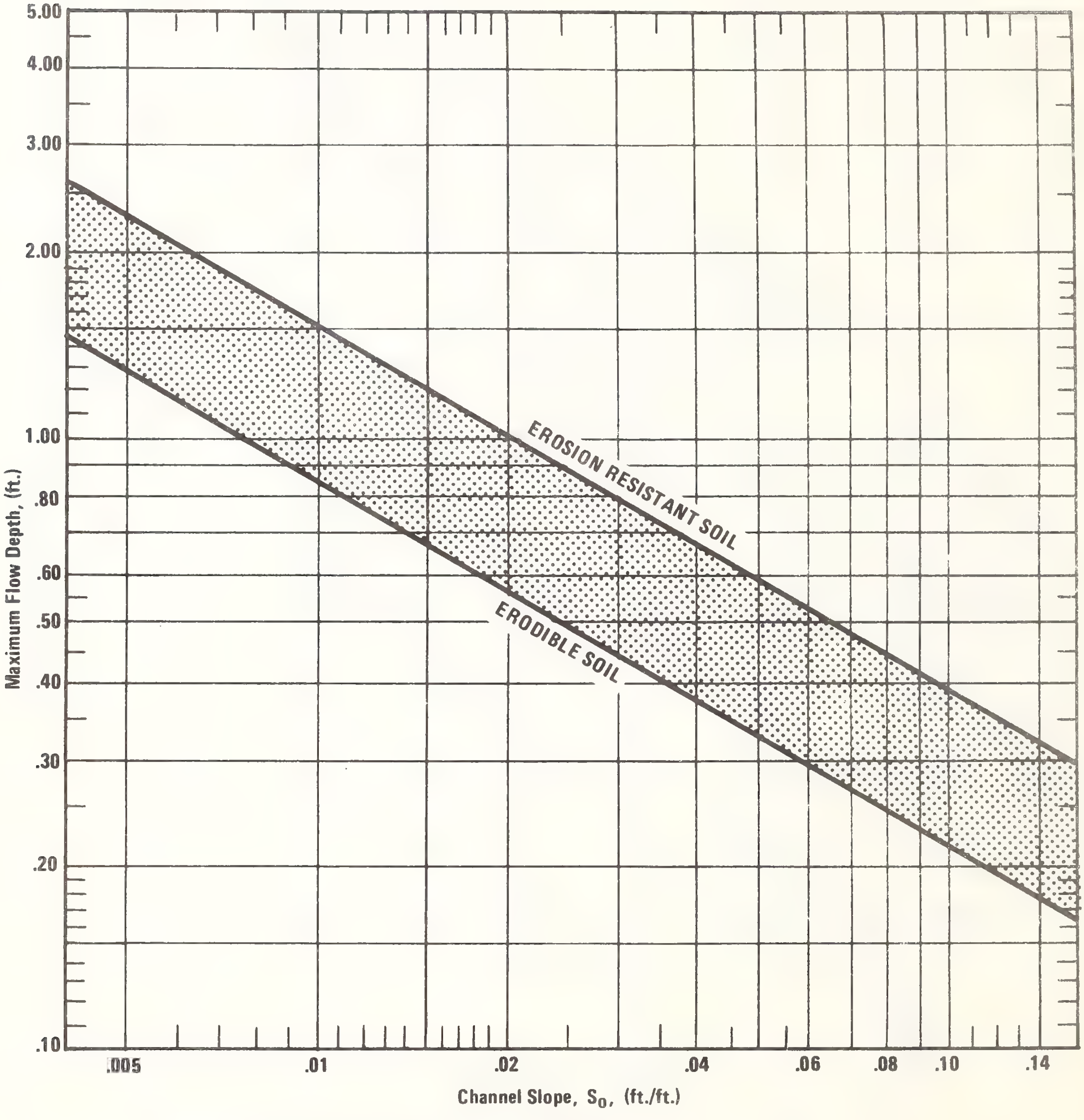
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH JUTE MESH



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

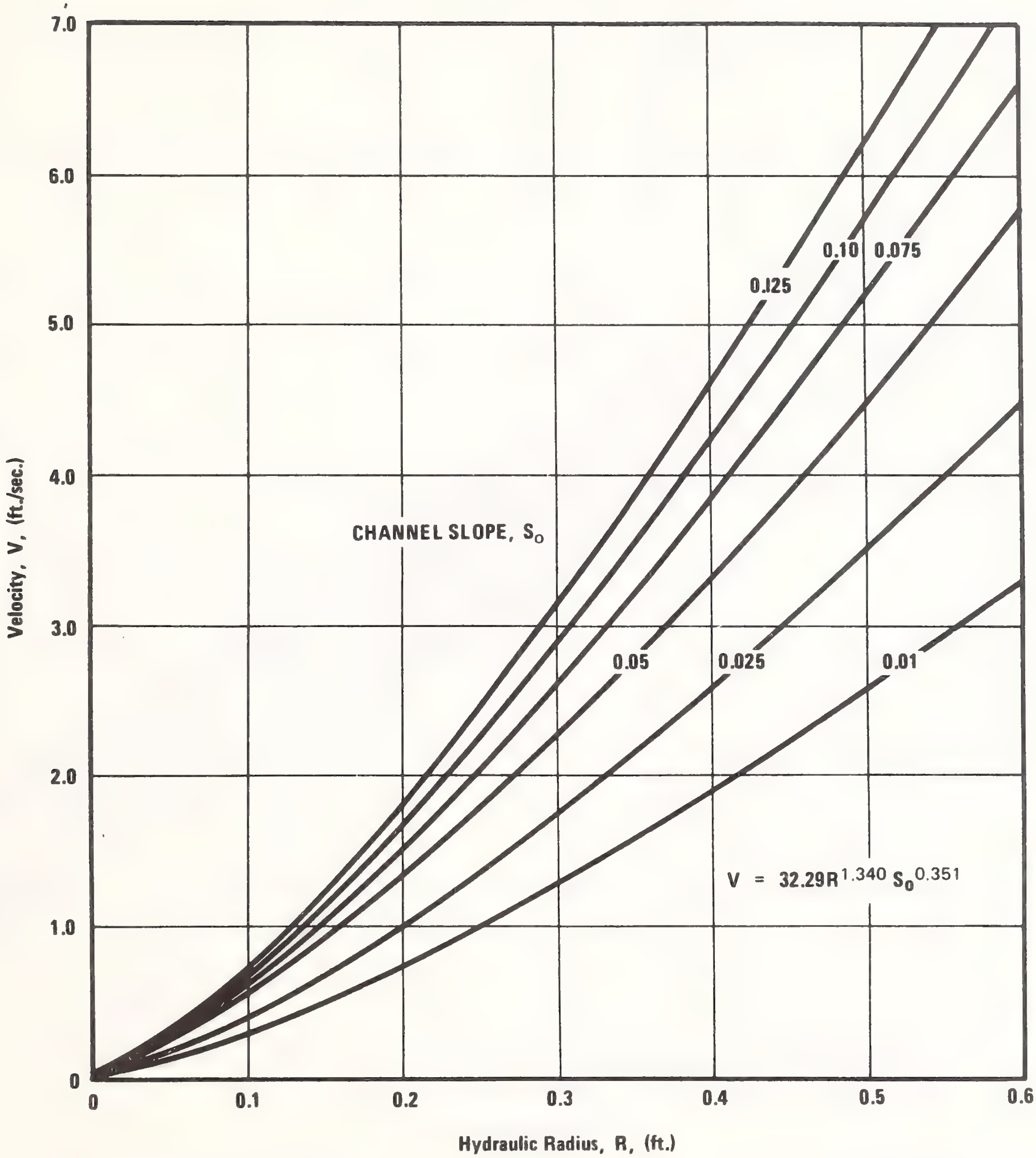
FLOW VELOCITY FOR CHANNELS LINED WITH JUTE MESH

Chart 4.134



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

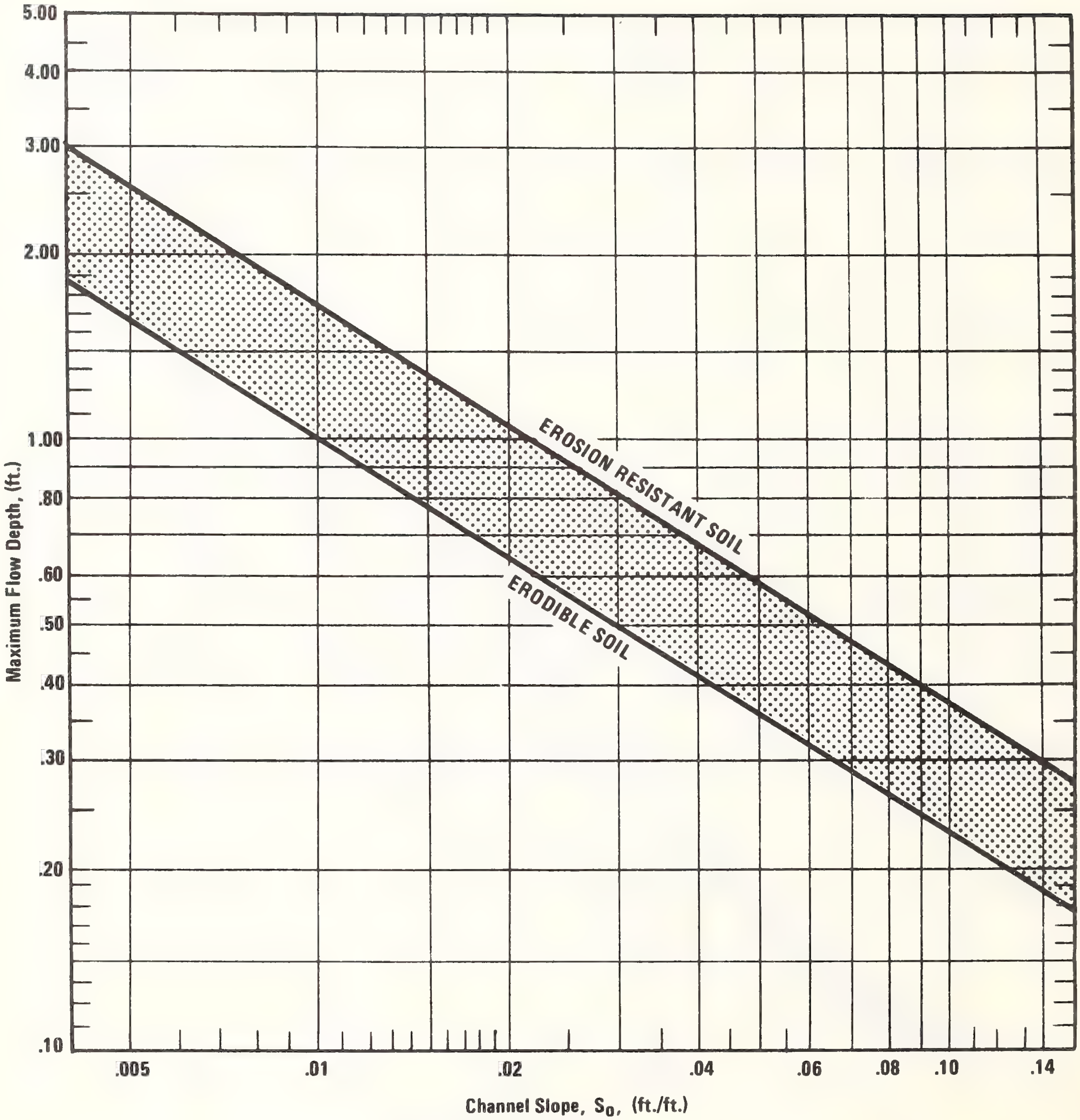
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH EXCELSIOR MAT



From Mississippi State University Report,
"Erosion Control Criteria for Drainage Channels"

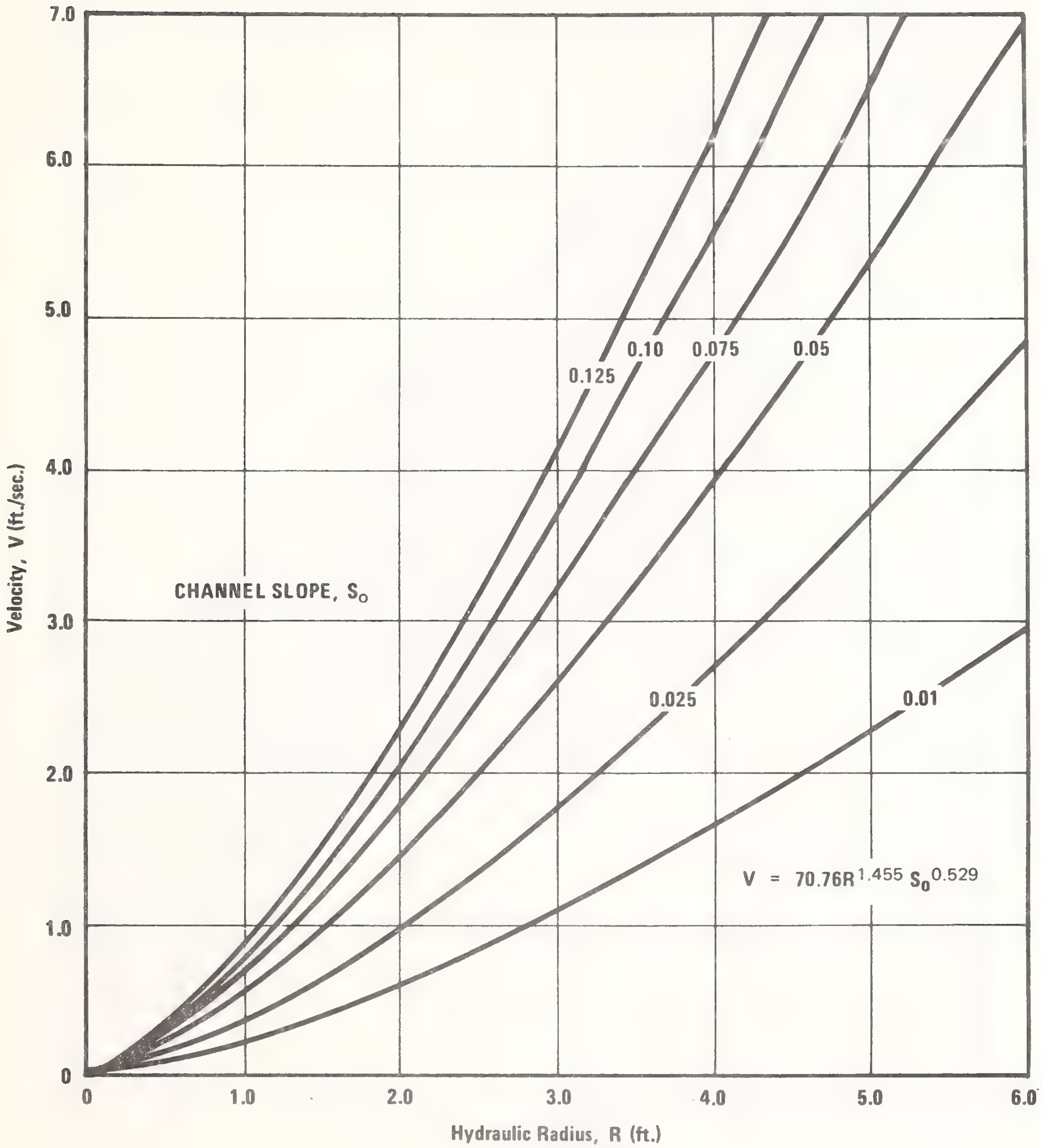
FLOW VELOCITY FOR CHANNELS LINED WITH EXCELSIOR MAT

Chart 4.136



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

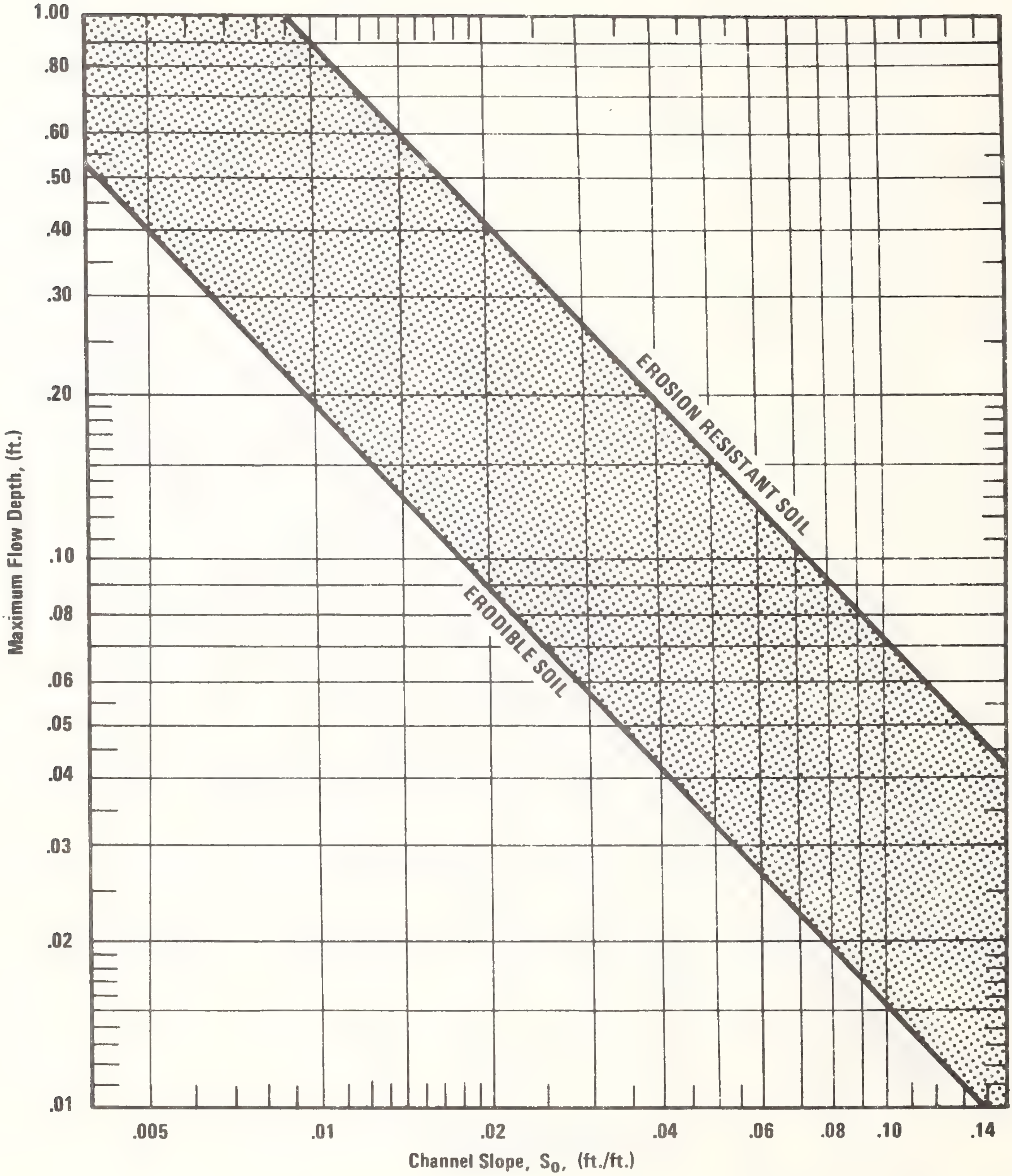
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH STRAW AND EROSIONET



From Mississippi State University Report,
"Erosion Control Criteria for Drainage Channels"

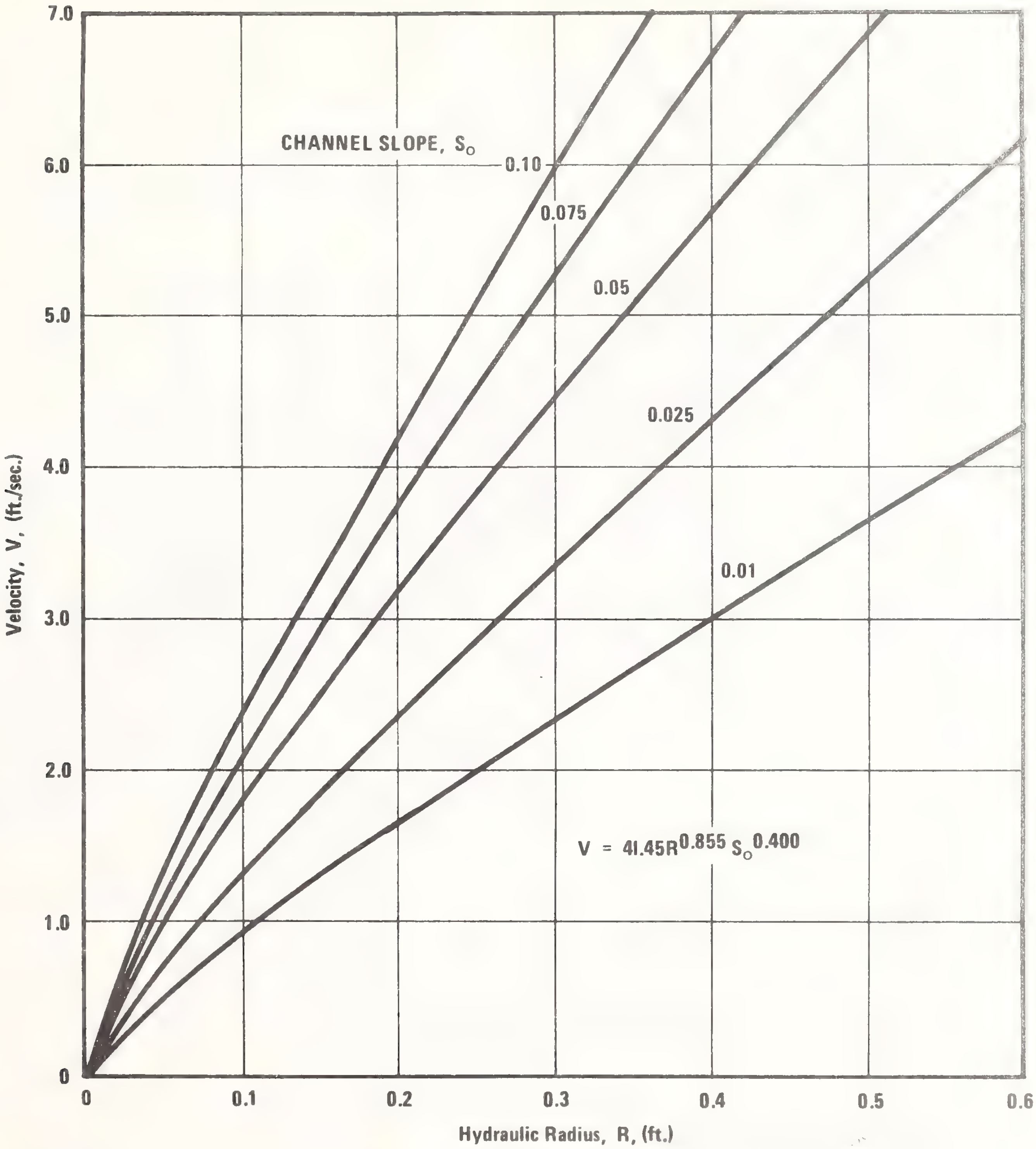
FLOW VELOCITY FOR CHANNELS LINED WITH STRAW WITH EROSIONET

Chart 4.138



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

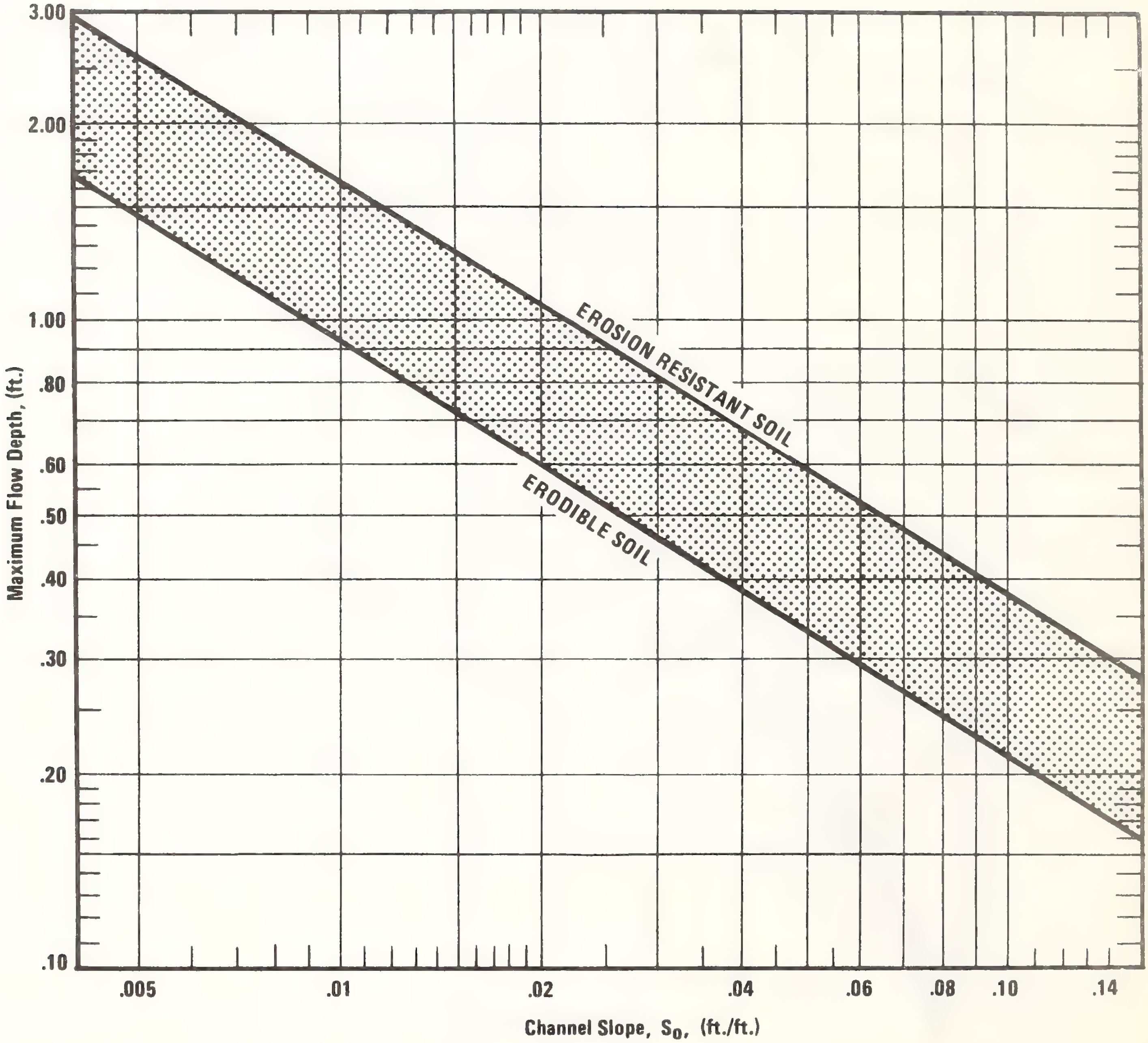
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH EROSIONET



From Mississippi State University Report,
"Erosion Control Criteria for Drainage Channels"

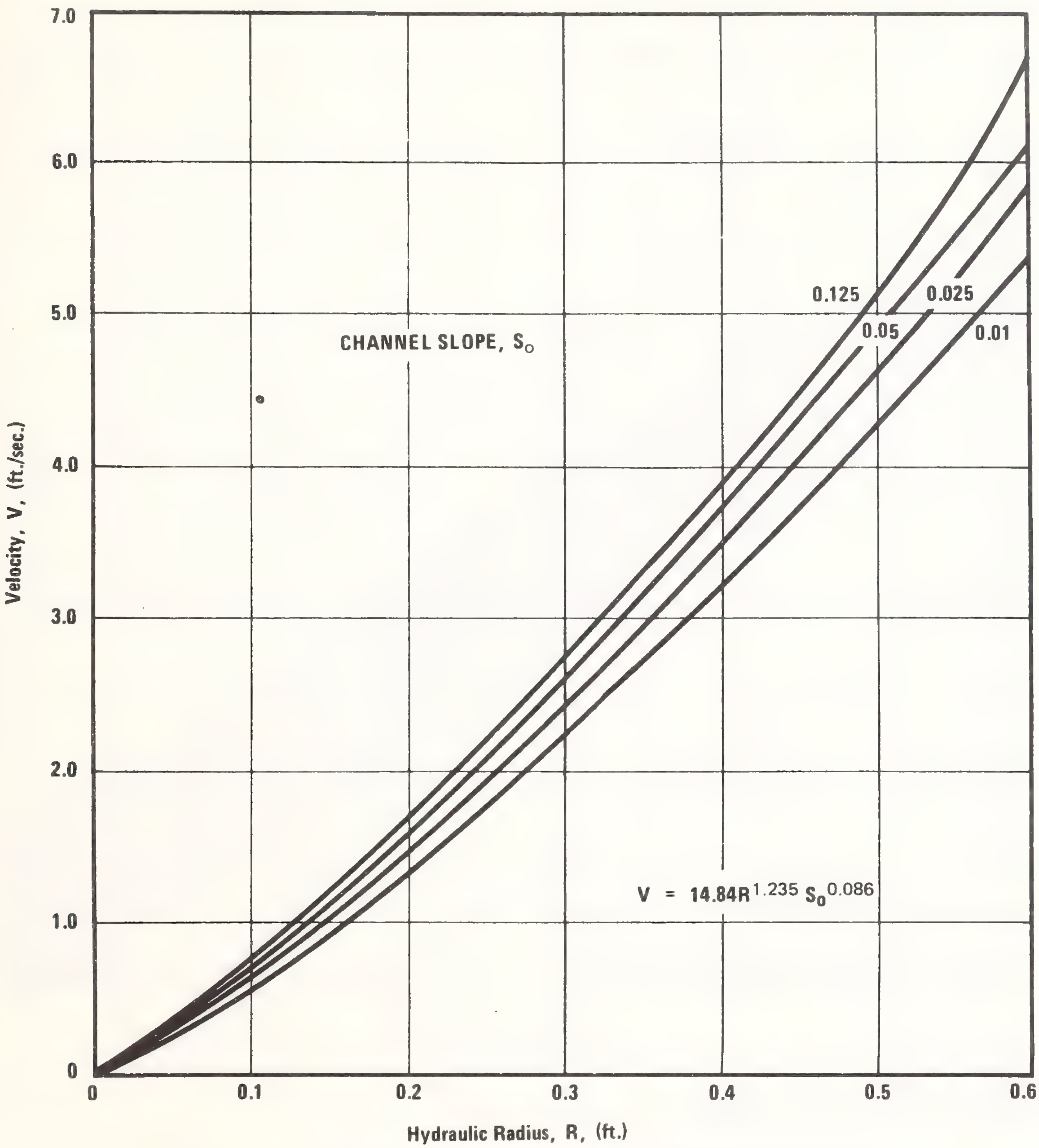
FLOW VELOCITY FOR CHANNELS LINED WITH EROSIONET

Chart 4.140



From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

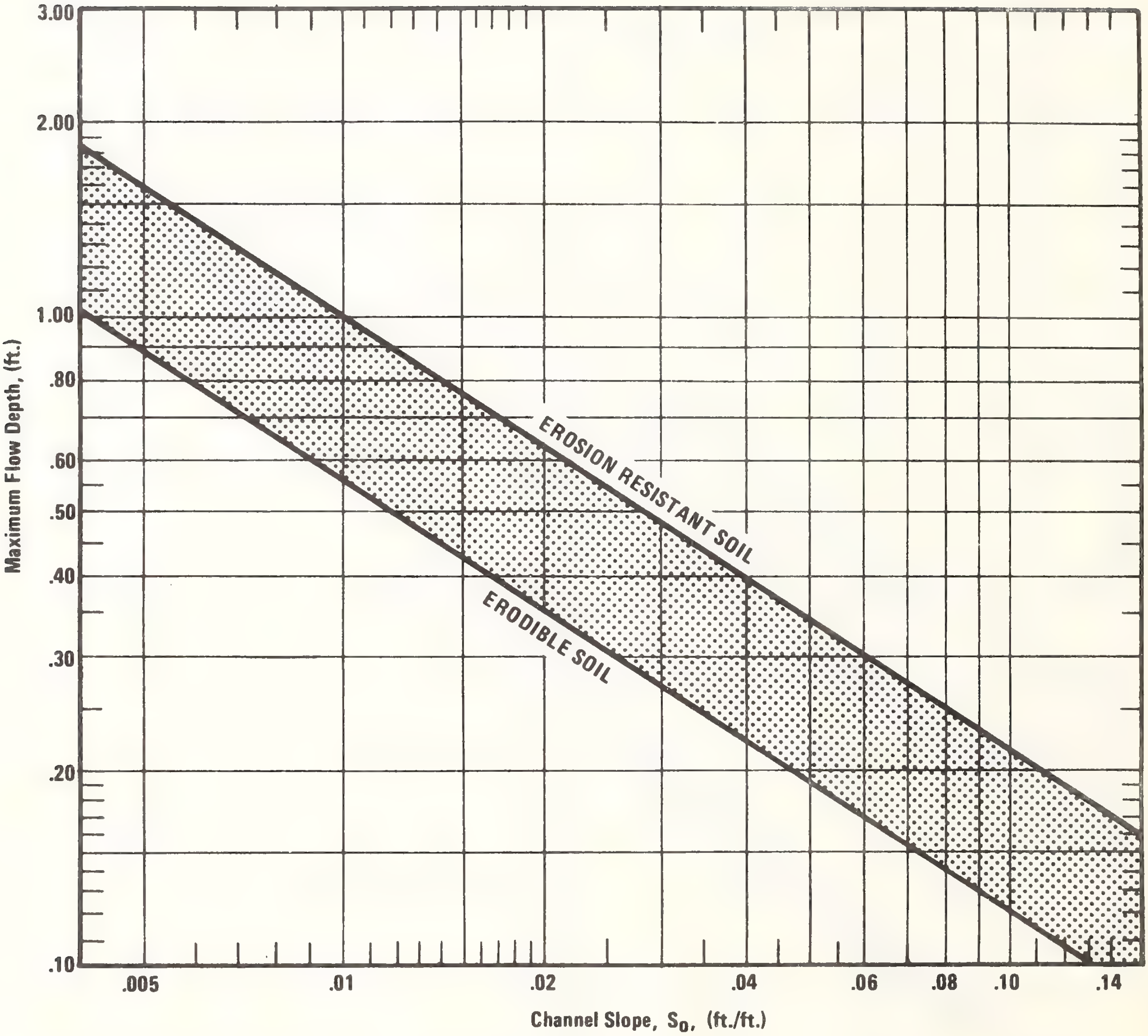
MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH 1/2" FIBERGLASS MAT



From Mississippi State University Report,
"Erosion Control Criteria for Drainage Channels"

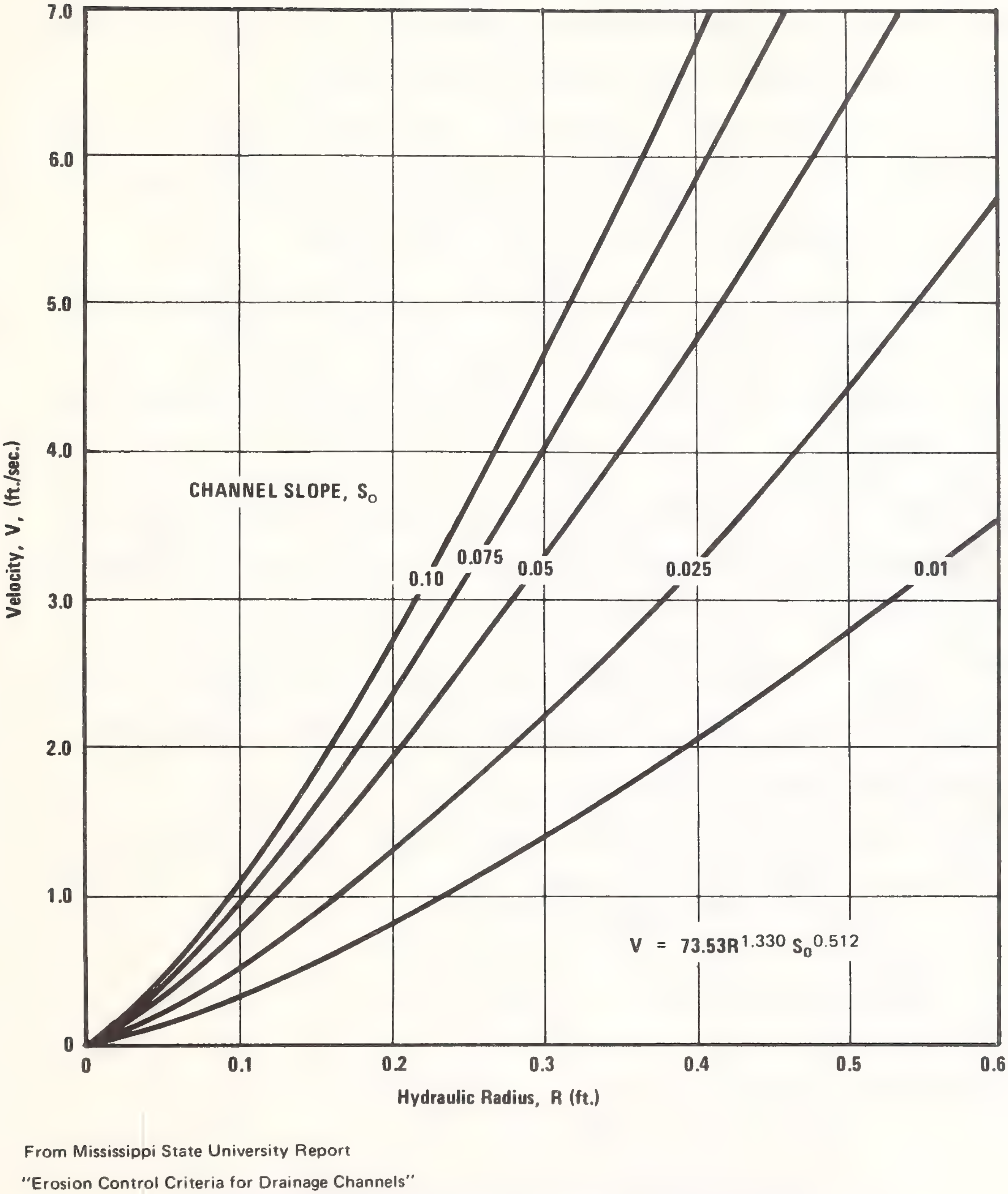
FLOW VELOCITY FOR CHANNELS LINED WITH 1/2 INCH FIBERGLASS MAT

Chart 4.142



From Mississippi State University Report,
" Erosion Control Criteria for Drainage Channels"

MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH 3/8" FIBERGLASS MAT



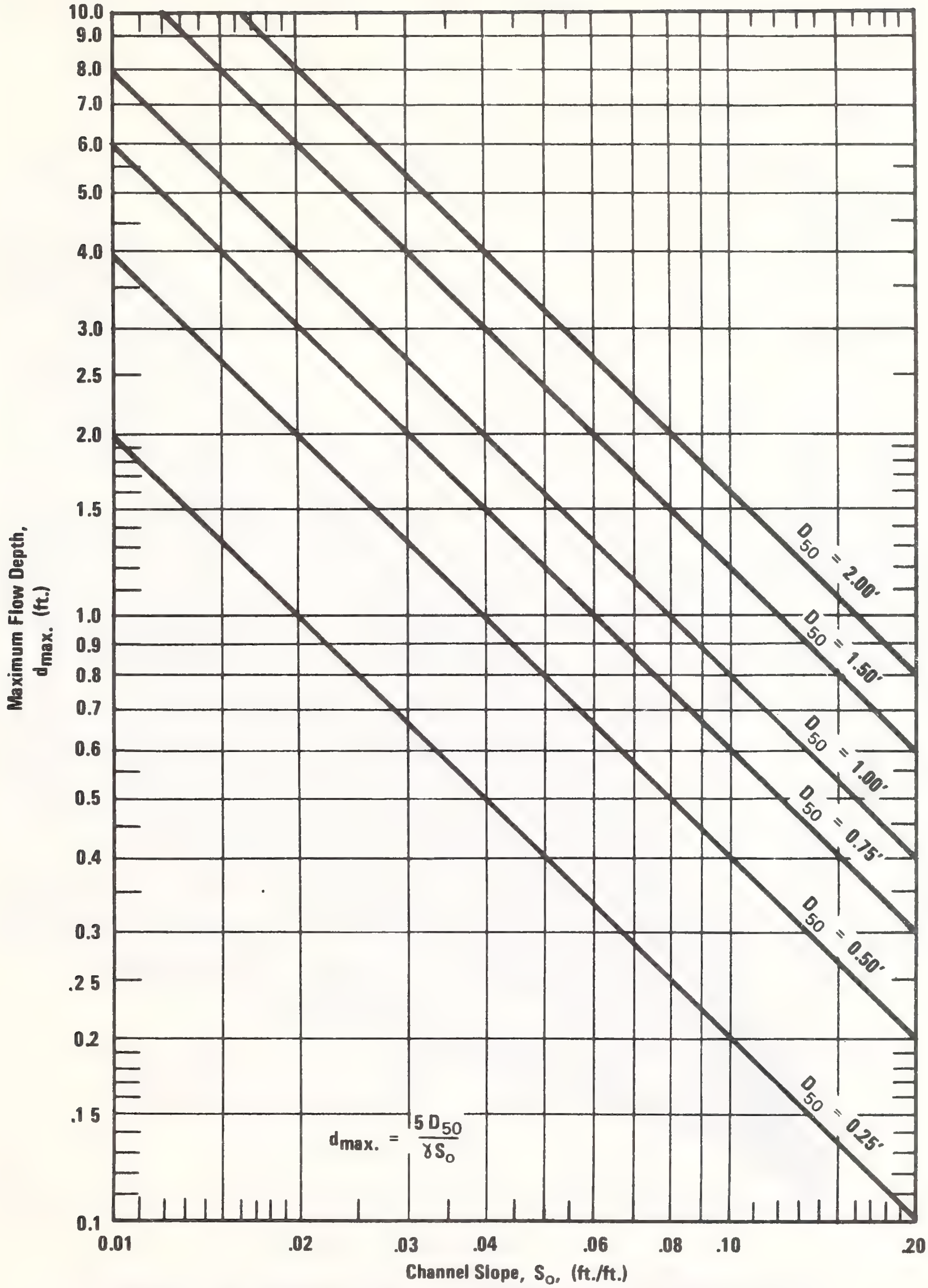
From Mississippi State University Report
"Erosion Control Criteria for Drainage Channels"

FLOW VELOCITY FOR CHANNELS LINED WITH 3/8 INCH FIBERGLASS MAT

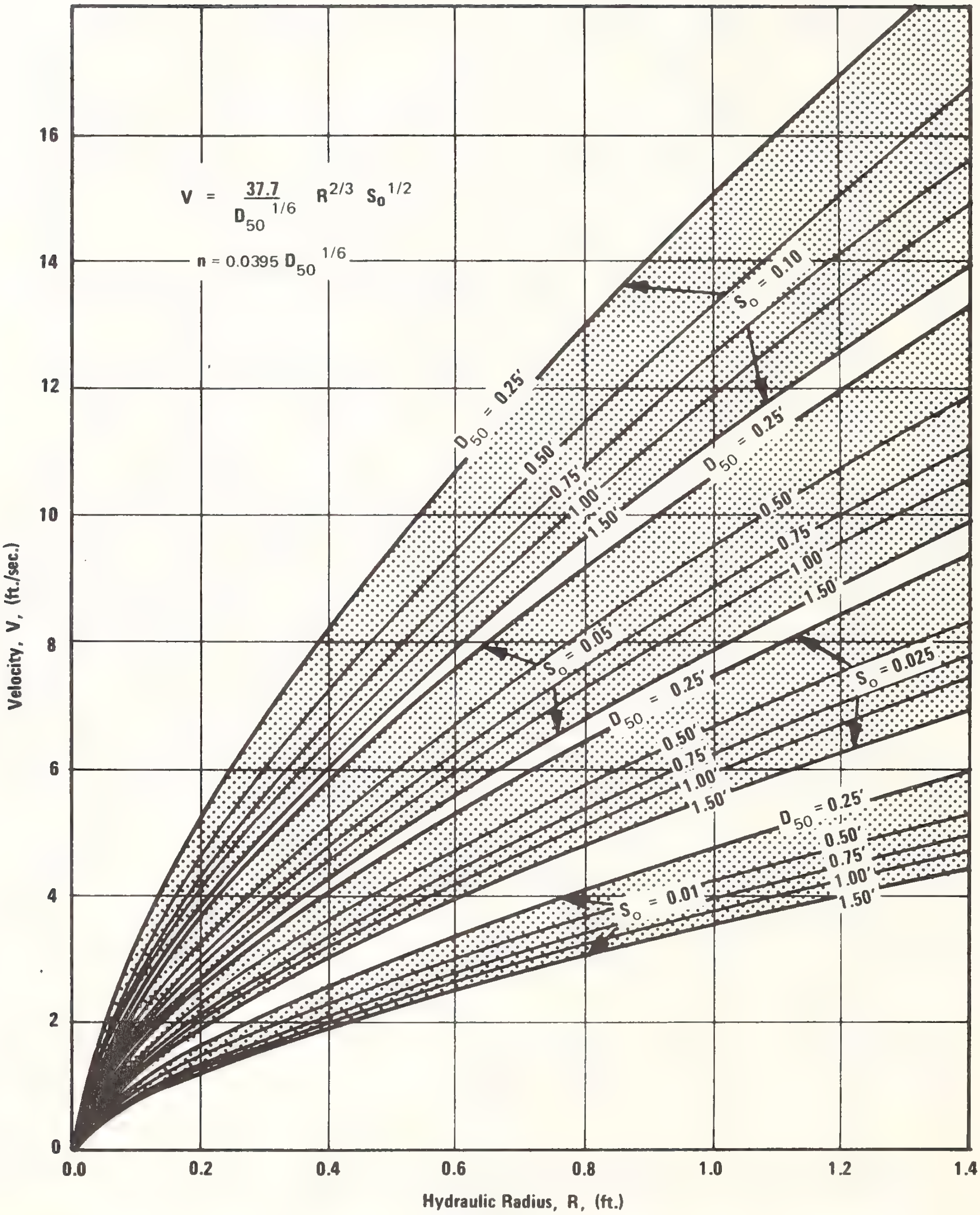
Rip Rap Lining - The design procedure for rock rip rap channel linings was developed as a part of National Cooperative Highway Research Program study under the sponsorship of AASHTO in cooperation with the Federal Highway Administration. The rip rap design procedures outlined here should be used only for channel linings. Rip rap to be used where flow is markedly nonuniform, or where the flow should be designed by the procedures of Section 4.81.

When channel side slopes are steeper than 3:1, and rock rip rap is chosen as the channel lining, the channel sides may become unstable. To design rip rap for the channel sides, use the following procedure:

1. Determine the size of rock required for the channel bottom from the procedure outlined previously and Charts 4.144 and 4.145.
2. From Chart 4.146, determine the angle of repose for the bottom rock size and shape.
3. From Chart 4.147, determine K_1, the ratio of maximum side shear to maximum bottom shear for a trapezoidal channel, based on B/d and side slope, Z.
4. From Chart 4.148, determine K_2, the ratio of critical shear on the side to critical shear on the channel bottom, based on side slope and the stone angle of repose.
5. The required D_{50} for the side slopes is K_1/K_2 times D_{50} for the bottom. (D_{50}) sides $= \frac{K_1}{K_2} (D_{50})$ bottom.

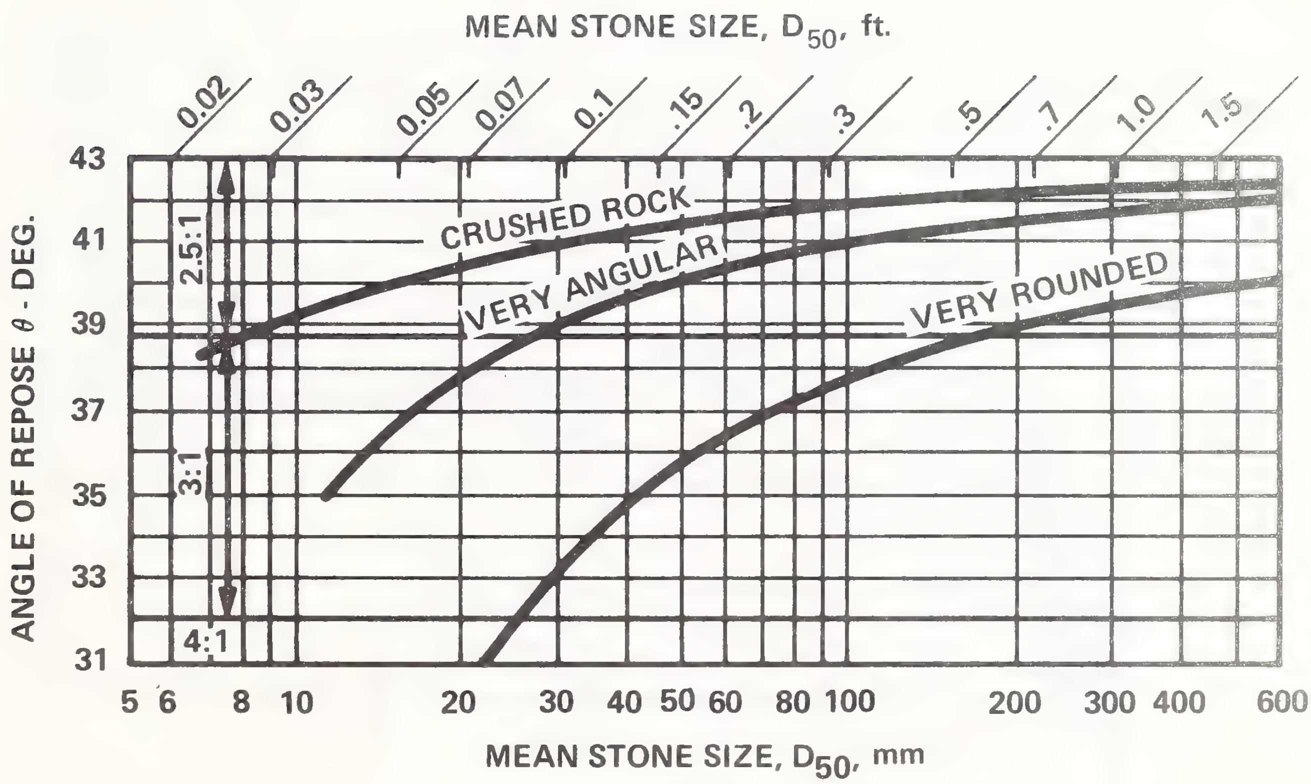


MAXIMUM PERMISSIBLE DEPTH OF FLOW (d_{max}) FOR CHANNELS LINED WITH ROCK RIPRAP

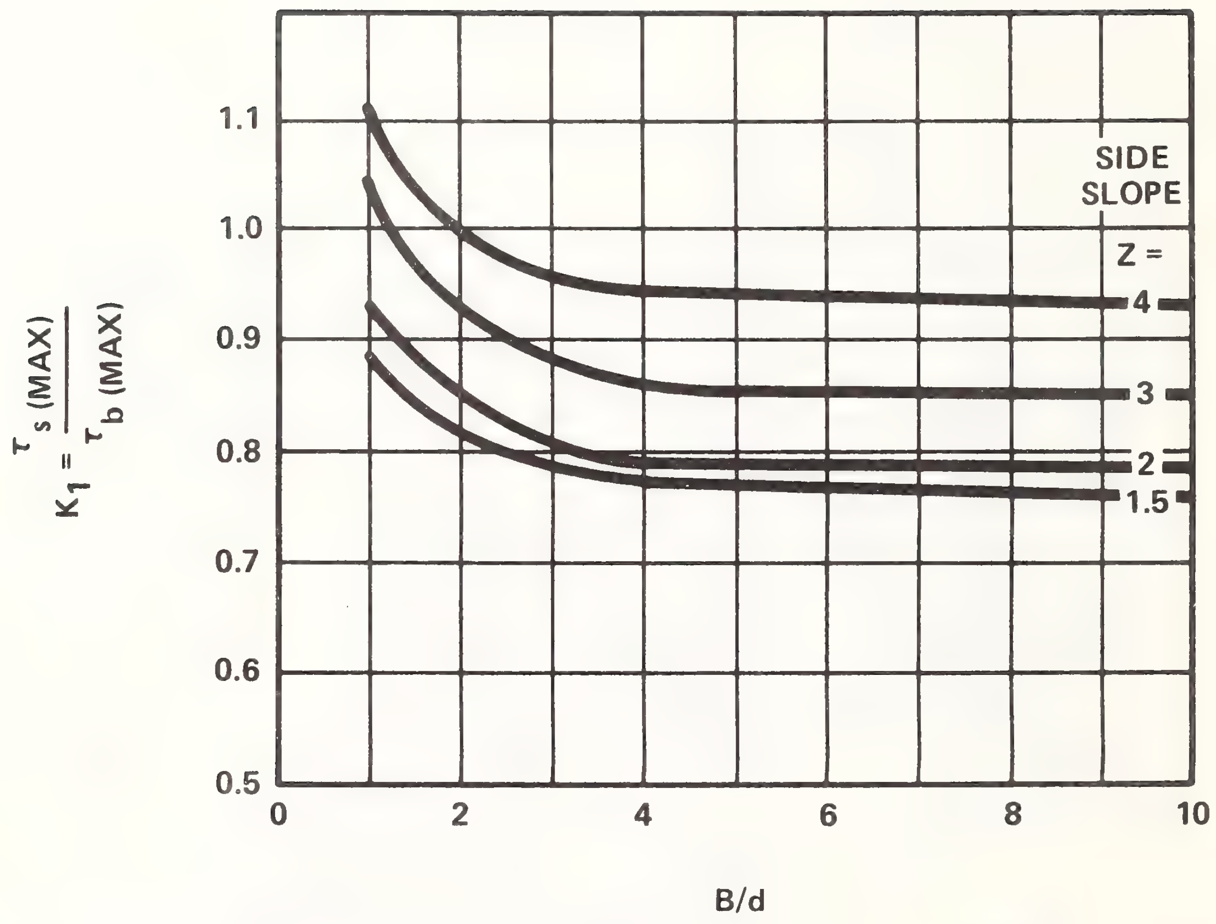


FLOW VELOCITY FOR CHANNELS LINED WITH ROCK RIPRAP
SLOPES=0.01 TO 0.10, D_{50} = 0.25' TO 1.50'

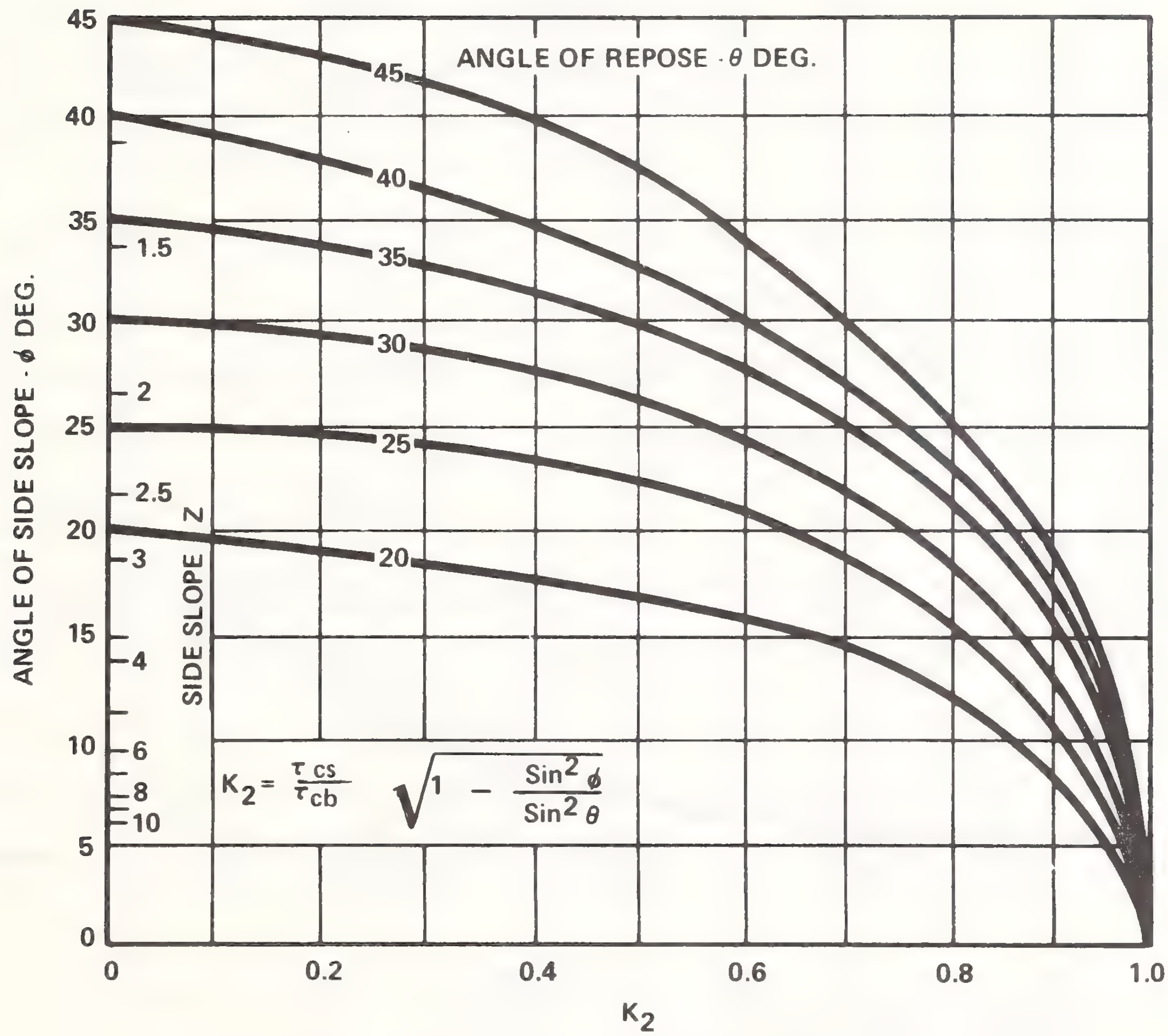
Chart 4.146



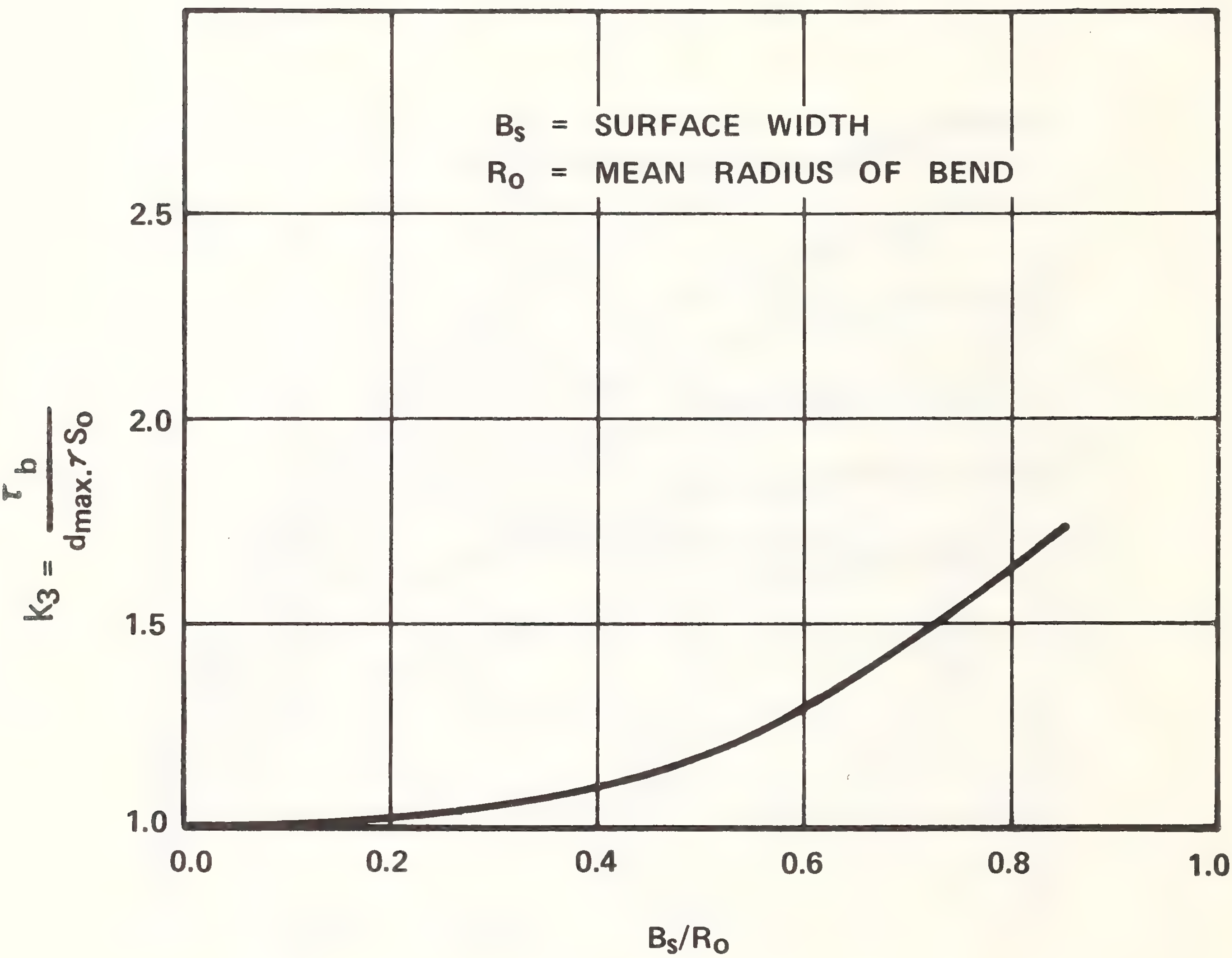
Angle of Repose of Riprap in Terms of Mean Size and Shape of Stone.



Distribution of Boundary Shear Around Wetted Perimeter of Trapezoidal Channels



Ratio of Critical Shear on Sides to Critical Shear on Bottom for Noncohesive Sediment.



RATIO OF MAXIMUM BOUNDARY SHEAR IN BENDS TO MAXIMUM BOTTOM SHEAR IN STRAIGHT REACH

Example Problem No. 1:

The following example problem was formulated to illustrate the use of the design charts and the concepts involved.

The objective was to design a channel lining for a trapezoidal channel with a 4 foot bottom width and 4:1 side slopes. Based on an analysis of the risks of channel failure, it was decided to design the permanent lining for a 10 year recurrence interval runoff and the temporary lining for the mean annual flow, or a recurrence interval of 2.33 years.

To determine the runoff ratio, the Rational Equation was used for the 4.3 acre drainage area. The soil was judged to have an average erodibility. Due to the right-of-way constraints, the channel top width must be restricted to 12 feet. Channel slope is 5 percent. Several permanent and temporary channel lining materials were available.

Detailed calculations are shown in Chart 4.64.

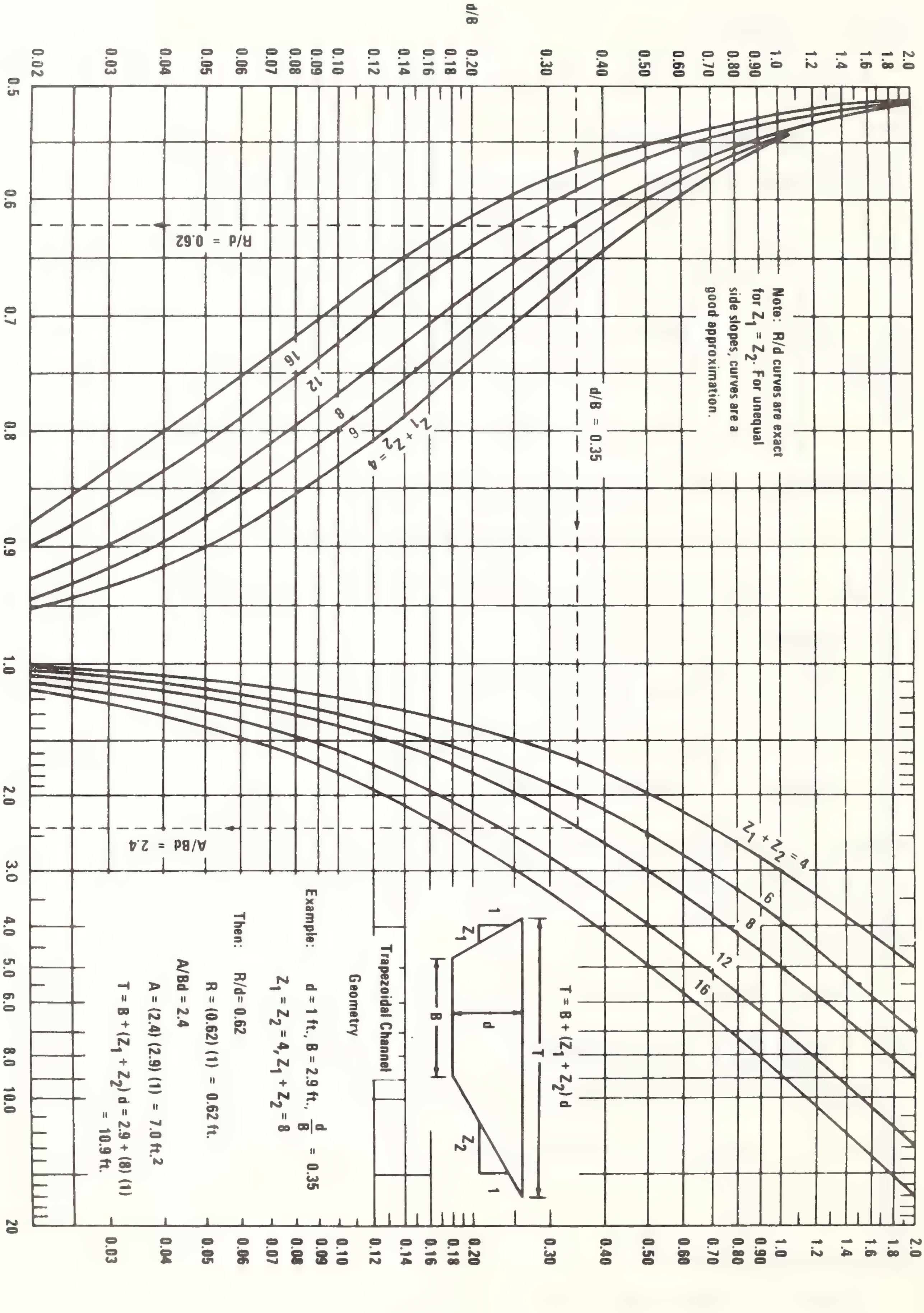
Note that the bare soil would convey very little flow on this 5 percent slope. Only rock rip rap was adequate. Had the 6 inch Alfalfa been adequate, temporary linings of either a double layer of fiber glass roving and asphalt or excelsior mat would have been adequate to convey the mean annual flow rate of 5.0 c.f.s.

The concrete lining has no d_{max}. It is computed that a 1.0 foot depth of flow in the concrete lining at a 5 percent slope would convey 154 cfs at a velocity of 19 ft./sec. This is the hydraulic advantage and disadvantage of a concrete lining in a nutshell: high capacity coupled with a high, erosive outlet velocity.

Example Problem No. 2:

Assume a trapezoidal channel with bottom width of 4 feet, permissible depth of flow of 4 feet, and side slopes of 1.5:1.

1. From design method $(D_{50})_{bottom} = 0.5$ feet. Available stone is classified as very angular.



Trapezoidal Channel Geometry

Fig. 4.64

Drainage Channel Lining Design

DATE: 11/9/72

PROJECT: FAI - I (87) DESIGNER: ____

STATION 105+40 TO STATION 112+80 CHECKED BY ____

DRAINAGE AREA = 4.3 ACRES DATE ____

HYDROLOGIC COMPUTATIONS:

10 year flow

$Q = CiA = (0.7)(5.0)(4.3) = 15$ cfs

mean annual flow

$Q = CiA = (0.7)(1.6)(4.3) = 4.8$ cfs

use 5 cfs

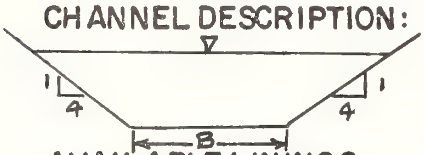
DESIGN FLOW: Q_{10} = 15 cfs

DESIGN FLOW FOR TEMPORARY LINING: $Q_{2.33}$ = 5 cfs

SOIL ERODIBILITY: Average

CHANNEL DESCRIPTION: MAX. TOPWIDTH = 12 ft. $T = B + (z_1 + z_2)d$

$= 4 + 8d$



S_0 = 0.05

AVAILABLE LININGS:

Permanent
1. Bare Soil
2. Alfalfa
3. RipRap (3", 6", 12")

Temporary
1. Fiber Glass Roving
2. Excelsior mat

LINING	d_{max}	B	$\frac{d_{max}}{B}$	$\frac{A}{Bd}$	A	$\frac{R}{d}$	R	V	Q=AV	T	REMARKS
Bare Soil	0.04	4	0.01	chart 4.150 —	0.17	chart 4.150 —	0.038	1.40	0.24	4.3	No Good
Alfalfa	0.85	4	0.21	1.85	6.29	0.67	0.57	0.92	5.8	10.8	No Good
Grass (retard)											
3" Riprap	0.40	4	0.10	1.4	2.24	0.78	0.31	4.9	11.0	7.2	No Good
6" Riprap	0.80	4	0.20	1.8	5.76	0.68	0.54	6.3	36.3	10.4	OK - over designed
Therefore, use 6" Riprap											
Temporary Linings											
Fiber Glass											
1 layer	0.16	4	0.04	1.16	0.74	0.87	0.14	2.5	1.85	5.3	No Good
2 layers	0.40	4	0.10	1.4	2.24	0.77	0.31	6.0	13.4	7.2	OK
Excelsior Mat	0.45	4	0.11	1.4	2.24	0.76	0.34	2.7	6.05	7.6	OK
Concrete	1.0*							Chart 4.151 19	154		High Velocity
*No d_{max}											Rigid Channel

2. From Chart 4.146, θ = 42 degrees.

3. From Chart 1.147, with B/D = 1.0 and Z = 1.5, K_1 = 0.88.

4. From Chart 1.148, with Z = 1.5 and θ = 42 degrees, K_2 = 0.53.

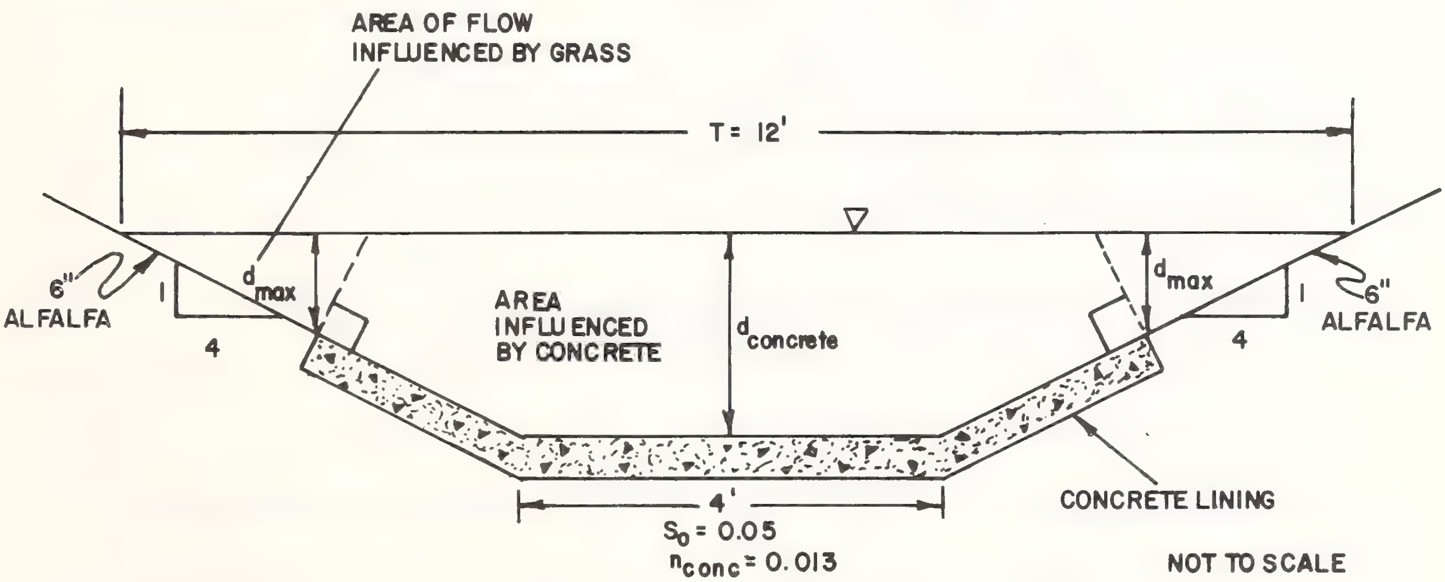
5. $(D_{50})_{\text{sides}} = \frac{0.88}{0.53} (D_{50})_{\text{bottom}}$

$$= \frac{0.88}{0.53} (0.5) = 0.83 \text{ ft.}$$

Caution: If the angle of the channel side slope exceeds the stone angle of repose the channel sides are unstable at any flow rate.

Example Problem No. 3:

Compute the capacity of the channel in Example Problem No. 1, if the top width is limited to 12 feet, and the channel bottom and part of the sides is concrete lined. S_0 = 0.05, B = 4 ft. $Z_1 = Z_2$ = 4. The alfalfa is to be mowed to a 6 inch length, on the average. The retardance is now "C".



The methods of this section may be used in the design of channels lined with more than one material. The most common composite channel is a grass lined channel with a concrete liner in the channel bottom. The smooth concrete liner greatly increases the capacity of the channel, while the grass lined channel sides provide freeboard.

The junction of two materials with different resistance to flow is critical due to the development of secondary currents at the shear zone. Such junctions

should be carefully installed, and d_{max} should be chosen conservatively.

d_{max} (Alfalfa) = 0.50 ft. (Chart 4.122)

Use lower (erodible) value to compensate for secondary currents.

For T_{max} = 12 ft., d (concrete) = 1.0'

$A_{total} = (4.0)(1.0) + 4(1.0)^2 = 8.0 \text{ ft.}^2$

$P_{total} = 4.0 + 2(\sqrt{17})(1.0) = 12.25$ ft.

For shear boundary, take normal to side slopes at edge of concrete lining.

Alfalfa = $(4\, d_{max} + 0.25\, d_{max})$ Grass d_{max} Grass = (4.25)

$(0.5)^2 = 1.06 \text{ ft.}^2$

Alfalfa = $2(\sqrt{17})\, d_{max} = 2(\sqrt{17})(.5) = 4.12$ ft.

$R_{Alfalfa} = \frac{1.06}{4.12} = 0.257$ ft.

$V_{Alfalfa}$ = .2 ft/sec. (Chart 4.125) (Use as lower limit)

$Q_{Alfalfa} = AV = (1.06)(.2) = 0.21$ cfs.

$A_{Concrete} = A_{Total} - A_{Grass} = 8.0 - 1.06 = 6.94 \text{ ft.}^2$

$P_{Concrete} = P_{Total} - P_{Grass} = 12.25 - 4.12 = 8.13$ ft.

$R_{Concrete} = \frac{6.94}{8.13} = 0.854$ ft.

$$Q_{Concrete} = \frac{1.486}{n} AR^{2/3} S_o^{1/2} = \left(\frac{1.486}{0.013}\right)(6.94)(0.854)^{2/3}(0.5)^{1/2}$$

$$= 159.7 \text{ cfs}$$

Q_{Total} = 159.7 + .2 = 159.9 (say 160), or about the same as the totally lined channel.

Outlet velocity $= \frac{Q_{Total}}{A_{Total}} = 160 = 20$ ft./sec.

4.83 SPUR DIKES

Where approach embankments encroach on wide flood plains and constrict the normal flood flow, special attention should be given to scour, particularly in the vicinity of bridge abutments. Flow from the flood plain travels along the embankment, and enters the constriction as a concentrated jet normal to the direction of flow in the main channel. In so doing the severity of the contraction is increased at the abutment, the effective length of bridge opening is reduced, and the possibility of scour at the junction of the two jets is great due to the violent mixing action.

This condition can be alleviated to some extent on new bridges by prohibiting borrow pits on the upstream side of embankments and forbidding the cutting of trees back of the toe of the fill slope. For cases where channeling along an embankment is already present or cannot be avoided, the situation can usually be remedied by constructing a spur dike as shown in Figure 4.65.

Where approach embankments divert considerable flood plain flow through the bridge opening, a spur dike, properly proportioned, is effective in reducing the gradient and velocity along the embankment by moving the mixing action of the merging flow away from the abutment to the upstream end of the dike. The combined flow is directed so that the entire waterway under the bridge is utilized and the depth of scour in the vicinity of the bridge abutment and at adjacent piers is reduced. Scour, if it occurs, is moved upstream away from the bridge structure. Although any spur dike is usually helpful in reducing scour from merging flood plain flow, a dike of proper proportions is needed to keep scour at the bridge abutment to a minimum and properly align the flow through the end spans of the bridge.

Three principal considerations are involved in proportioning a spur dike: geometry, height, and length. Laboratory studies showed that a dike shaped in the form of a quarter of an ellipse, with ratio of major (length) to minor (offset) axes of 2.5:1 performed as well or better than any shape tested. The height of spur dike is based on anticipated high water. It should have sufficient

height and freeboard to avoid overtopping and be protected from wave action. With the exception of dikes constructed entirely of stone or earth dikes properly armored with graded stone facing, overtopping will usually result in serious damage or complete destruction of a dike because the difference in level across the dike is usually sufficient to produce erosive velocities. The remaining dimension, length of dike, will be considered in detail in the following paragraphs. It may be said, however, that since field information on the operation of spur dikes is meager, the tendency at present is to lean toward over design rather than under design.

The information for determining the length of spur dike was obtained from model studies performed at Colorado State University, field data collected by the U.S. Geological Survey during floods in the State of Mississippi, and field observations by D. E. Schneible during floods. The various parameters for determining spur dike length are identified in Chart 4.151.

The parameters are a spur dike discharge ratio, Q_f/Q_{100}, relating the flow over the left or the right flood plain to a specific portion of the flow under the bridge, a representative velocity adjacent to the abutment of the bridge, and the length of spur dike needed. The discharge ratio is shown as the ordinate, the length of dike as the abscissa, and the family of curves are for different values of the velocity, V_{n2}.

Definitions of the symbols used are:

Q = Total discharge of stream (C.F.S.)

Q_f = Lateral or flood plain flow (one side) (C.F.S.)

Q_{100} = Discharge in 100 feet of stream adjacent to abutment (c.f.s.)

b = Length of bridge opening (ft.)

A_{n2} = Water area under bridge referred to normal stage (sq.ft.)

$V_{n2} = \frac{Q}{A_{n2}}$ = Average velocity through bridge opening (f.p.s.)

$\frac{Q_f}{Q_{100}}$ = Spur dike discharge ratio

L_s = Top length of spur dike (measured as shown on Chart 4.151 (ft.)

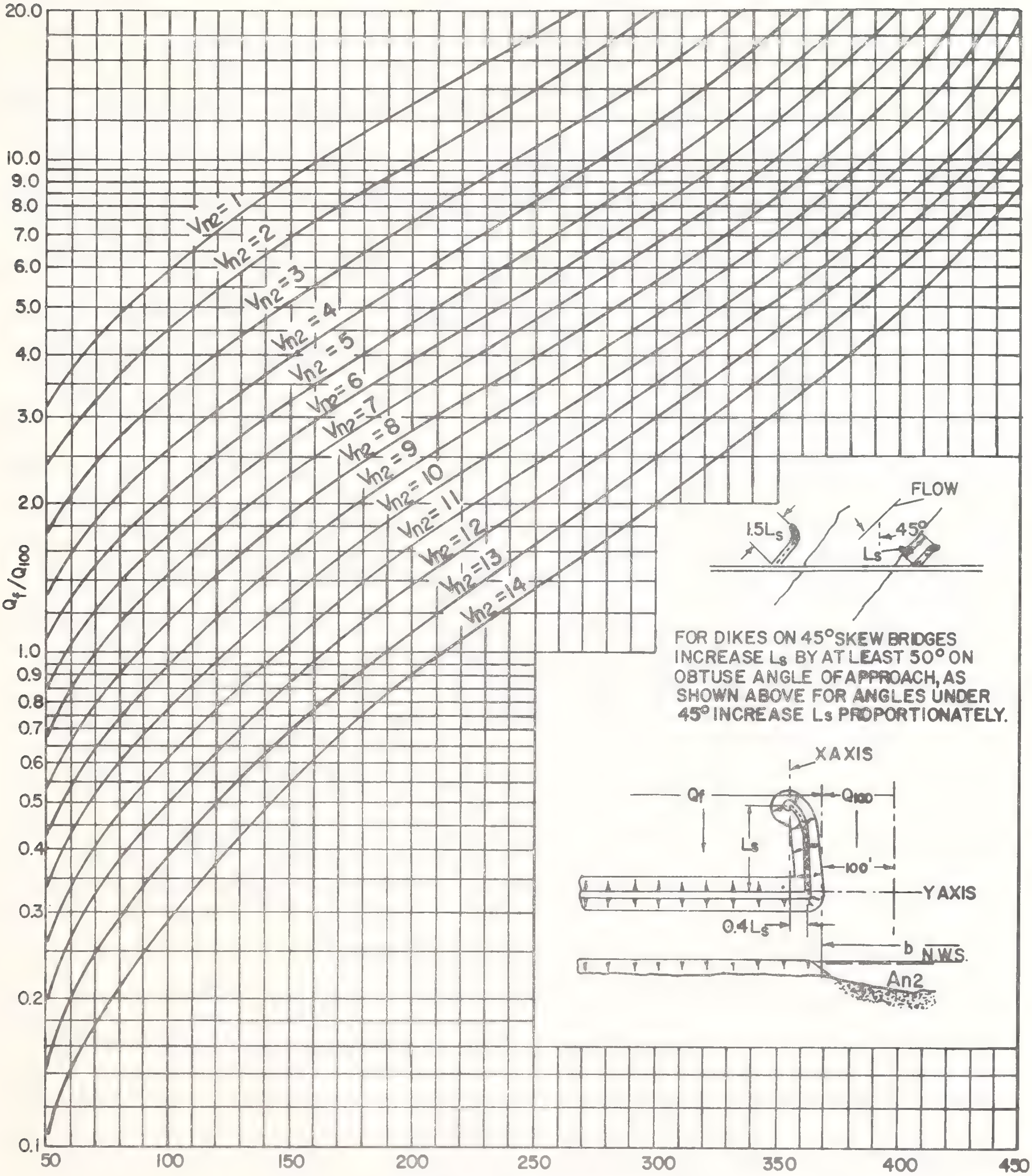


Chart for determining length of spur dikes.

The shape of the dike will conform to the equation of one quarter of an ellipse with 2.5:1 ratio of major to minor axis.

$$\frac{X^2}{L_s^2} + \frac{Y^2}{(0.4L_s)^2} = 1$$

or

$$L_s = (X^2 + 6.25Y^2)^{1/2}$$

It can be observed from Chart 4.151 that the length of a spur dike should be increased with an increase in flood plain discharge, with an increase in velocity under the bridge, or both. The chart is read by entering the ordinate with the proper value of Q_f/Q_{100}, moving horizontally to the curve corresponding with the computed value of V_{n2} and then downward to obtain from the abscissa the length of spur dike required. As a general rule, if the length read from the abscissa is less than 30 feet, a spur dike is not needed. For chart lengths from 30 to 100 feet, it is recommended that a spur dike no less than 100 feet long be constructed. This length is needed to direct the curvilinear flow around the end of the dike so that it will merge with the main channel flow and establish a straight course down river before reaching the bridge abutment. Curvilinear flow can have several times the capacity to scour than that of parallel flow, depending on the radius of curvature, velocity, depth of flow and other factors. Holding the depth of flow and other factors the same, the depth of scour will increase with decrease in radius of curvature. For this reason the deepest scour produced by a spur occurs near the nose where the radius of curvature is sharpest.

For bridges skewed at an angle of 45 degrees, it is recommended that the forward dike (see sketch, Chart 4.151) be lengthened by 50 percent over the value given by the design chart. For lesser angles, the forward dike may be lengthened in proportion. Chart 4.151 shows in detail a general plan and cross section of a spur dike as usually constructed.

Spur dikes may be constructed entirely of rock provided the facing is of sufficient size to resist displacement by the current. Dikes constructed of

earth should be compacted to the same standards as the roadway embankment and should extend above expected high water. Protection may be limited to the areas shown on Chart 4.151 if rock is expensive and the remaining portions of dike will support vegetation. Where rock is used as a facing on an earth dike, it should be well graded and a filter blanket should be used if the relative gradations of the rock and of the spur dike material require it. Design of filter blankets and rip rap protection are described in Section 4.81. In special cases where the cost of facing for a spur dike is prohibitive, it can be constructed with a sod cover or minimum protection with a plan for repair or replacement after each high water occurrence with the risk that it would protect the bridge for one flood.

The following points should be kept in mind:

1. Keep trees as close to the toe of the spur dike embankment as construction will permit.
2. Do not allow the cutting of channels or the digging of borrow pits near spur dikes or along the upstream side of embankments.
3. If drainage is important, put small pipe through spur dike or embankment to drain pockets left behind dikes after flood recedes.

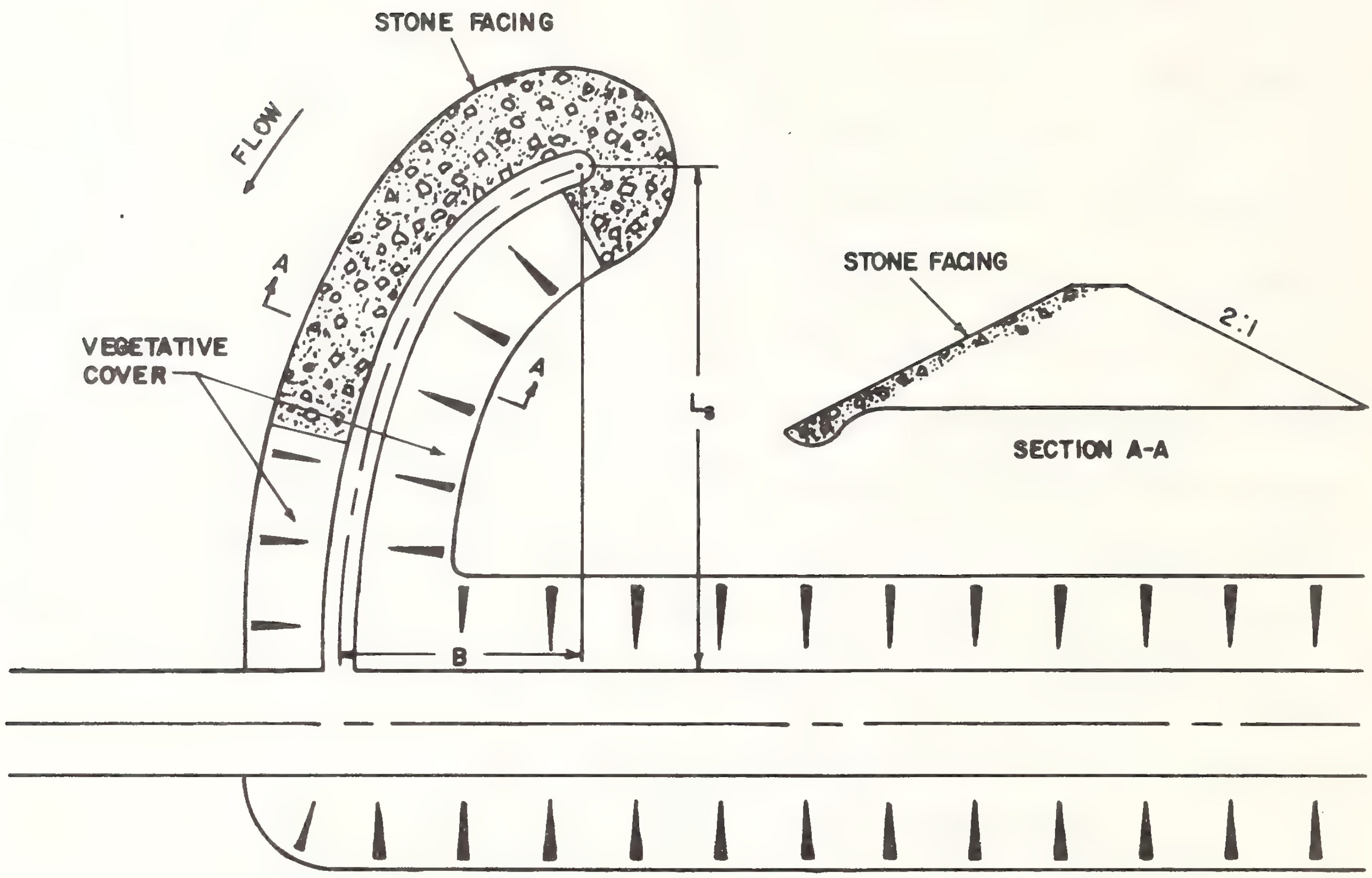


Figure 4.65-Plan and cross section of spur dike.

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SECTION 4.9
FLOOD ROUTING

4.9 FLOOD ROUTING METHOD OF CULVERT DESIGN

Introduction

It is sometimes possible in a culvert design problem to consider the highway fill as an earth dam, temporarily ponding the runoff, and discharging it slowly through a culvert spillway. This process is known as flood routing. The two major advantages of flood routing versus conventional culvert design are the savings realized due to the reduction in culvert size and the flood protection downstream resulting from the reduced flood peak.

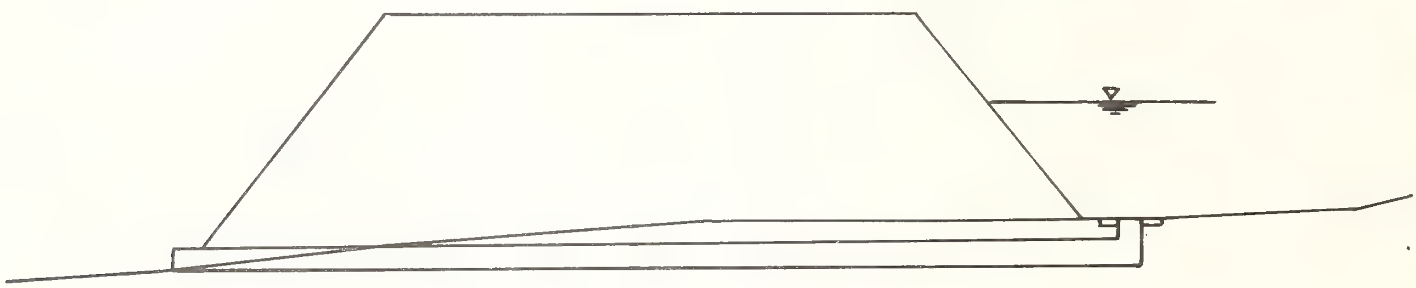
The flood routing design procedure should only be used for small (less than 10 square miles), semi-arid drainage areas subjected primarily to convective storms and does not have application in regions deriving their annual floods primarily from snow melt. A relatively high fill is usually required to provide the necessary storage.

4.91 ELEMENTS OF FLOOD ROUTING

In any flood routing procedure the factors that must be considered are (1) inflow, (2) outflow, and (3) storage.

The inflow is represented by an inflow hydrograph which defines the rate of flow as a function of time. Section 3.6, Synthetic Hydrograph Development, presents the procedure for developing an inflow hydrograph.

The outflow is represented by stage-discharge curves which define the rates of flow as functions of the size and type of discharge pipe, inlet configuration, and depth of water above the inlet (stage or head). Figure 4.66 shows a culvert spillway configuration that is used by the Soil Conservation Service which has worked well in this type of installation. The discharge values for the stage-discharge curve can be calculated using the charts and methods presented in Section 4.11, "Roadway Culverts".



CULVERT SPILLWAY
Figure 4.66

The storage is represented by a depth-capacity curve which defines the available storage capacity of the reservoir as a function of the depth (or stage) of the water. The capacity of the reservoir can be calculated by the average end area method with the areas scaled from a contour map. In most cases field personnel must be requested to make the contour map of the proposed site. Survey requests are discussed in Section 2.7, "Small Earth Dams".

Since the size of the culvert spillway cannot be determined directly, a size must be selected and the flood routed through the reservoir. If the peak

outflow rate, the maximum stage in the reservoir, or the detention time does not meet the design criteria, another size spillway must be selected and the flood routing procedure repeated. The flood routing procedure is outlined in the following section.

4.92 ROUTING PROCEDURE

The routing procedure involves the solution of the following routing equation:

$$t\left(\frac{i_1+i_2}{2}\right) = t\left(\frac{o_1+o_2}{2}\right) + S_2 - S_1$$

where i = inflow rate

o = outflow rate

S = volume of storage

t = time interval

1 and 2 = subscripts denoting beginning and end of the time interval, respectively

The solution of this equation involves a trial and error process which is beyond the scope of this manual. A hydrology or water resources textbook should be consulted if a detailed explanation of the procedure is required. The computer program entitled "FLD-IPLI" will make the necessary computations and should be used for the flood routing process. A discussion of the program and a sample problem are given in Section 4.94.

4.93 OTHER CONSIDERATIONS

When using the routing procedure to determine a culvert size, there are several other items that should be given adequate consideration.

Because it is not possible to provide an emergency spillway across the highway, an adequate factor of safety should be included in the design to insure that the water never overtops the roadway. A general guideline to follow is that the culvert spillway should be able to pass two of the design storms in succession without the water overtopping the roadway.

Erosion control should be provided for at the inlet and outlet ends of the culvert spillway. Various methods of erosion control are discussed in Section 4.8. If the inlet is placed immediately adjacent to the fill, the vortex may create additional erosion problems and should be controlled. The steel plate shown in Figure 4.67 will help reduce the vortex.

A trash rack may be required to prevent the culvert spillway from becoming plugged. A typical trash rack is also shown in Figure 4.67. Section 4.15 discusses debris control in more detail.

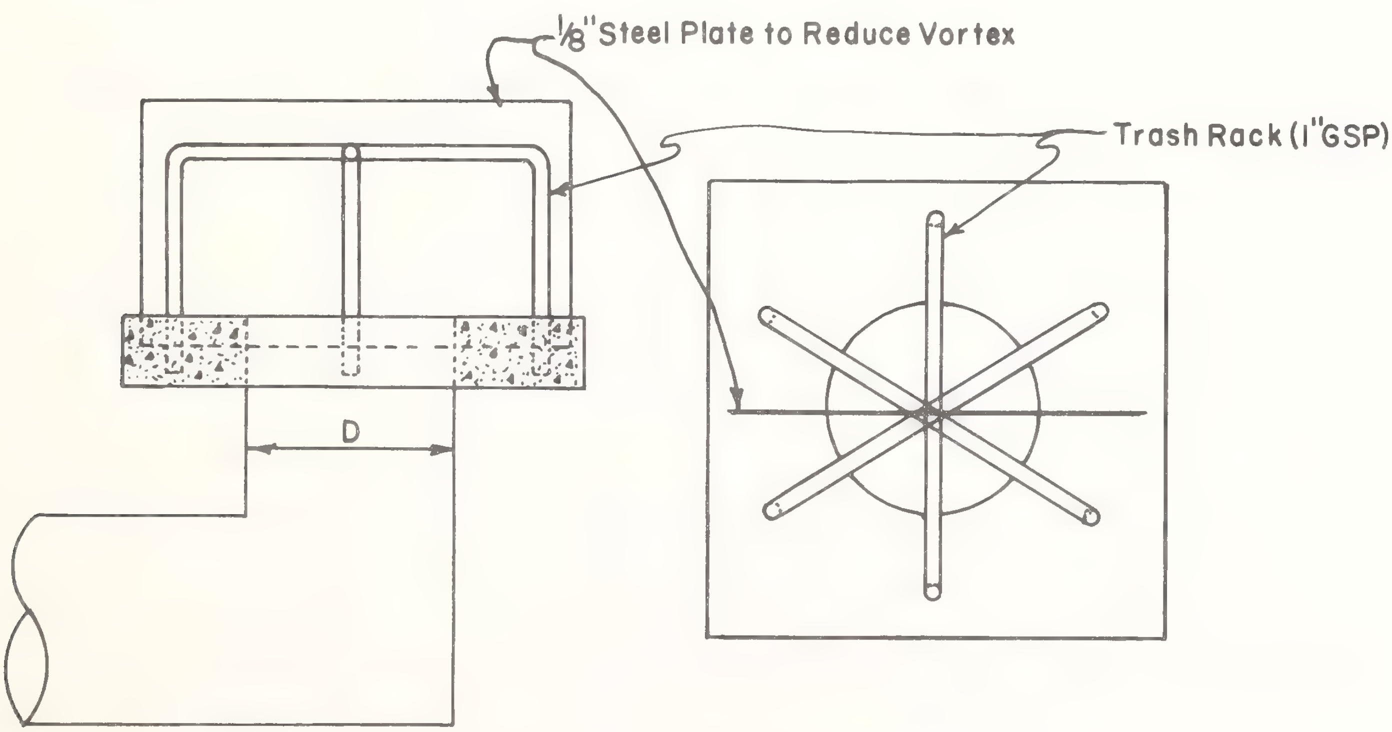


Figure 4.67 INLET PROTECTION

If the water is to ponded for any length of time, the Materials Bureau should be consulted to see if seepage could damage the fill. A seepage collar and/or cutoff trench may be required.

4.94 FLOOD ROUTING COMPUTER PROGRAM AND EXAMPLE

The following description and example illustrate the use of the flood routing computer program.

Name: FLD Language: PL 1 Input Format: Run

Purpose: Performs Flood Routing Computations.

Required Input Data: No. of inflow points, inflow rate (cfs) at each time interval, number of stage-storage points, number of stage-discharge points, stage (ft)-storage (arce-ft) coordinates in pairs, stage (ft)-discharge (cfs) coordinates in pairs, time increment in hours.

Abstract: This program takes an inflow hydrograph and using the stage-storage and stage-discharge data routes it through the reservoir. The program prints the stage, inflow, outflow, and storage for each time interval from the time the storm starts until the reservoir is empty. The stage-storage and stage-discharge curves are input by a series of coordinates which describe the curve. The time interval should be short enough to adequately describe the inflow hydrograph.

Limitations: The maximum number of inflow points, stage-storage points, and stage discharge points is 30. The computations are based upon 2% error.

Example: For the given inflow hydrograph, stage-storage curve and stage-discharge curve, calculate the outflow hydrograph and the maximum stage.

A time interval of .5 hour seems to adequately describe the inflow hydrograph. The coordinates used to define the inflow hydrograph, stage-storage, and stage-discharge curves are shown on the curves of Figure 4.68 and Figure 4.69.

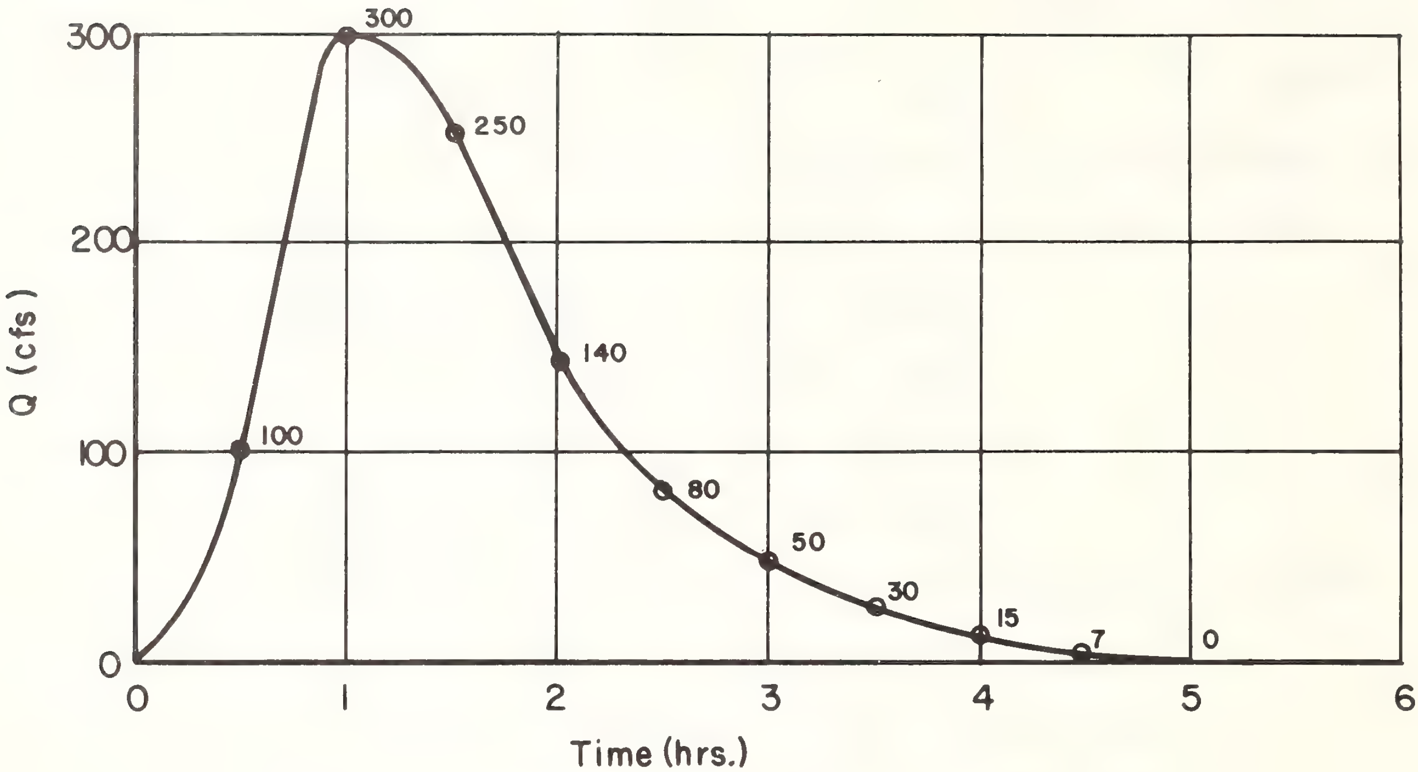


Figure 4.68 - Inflow Hydrograph

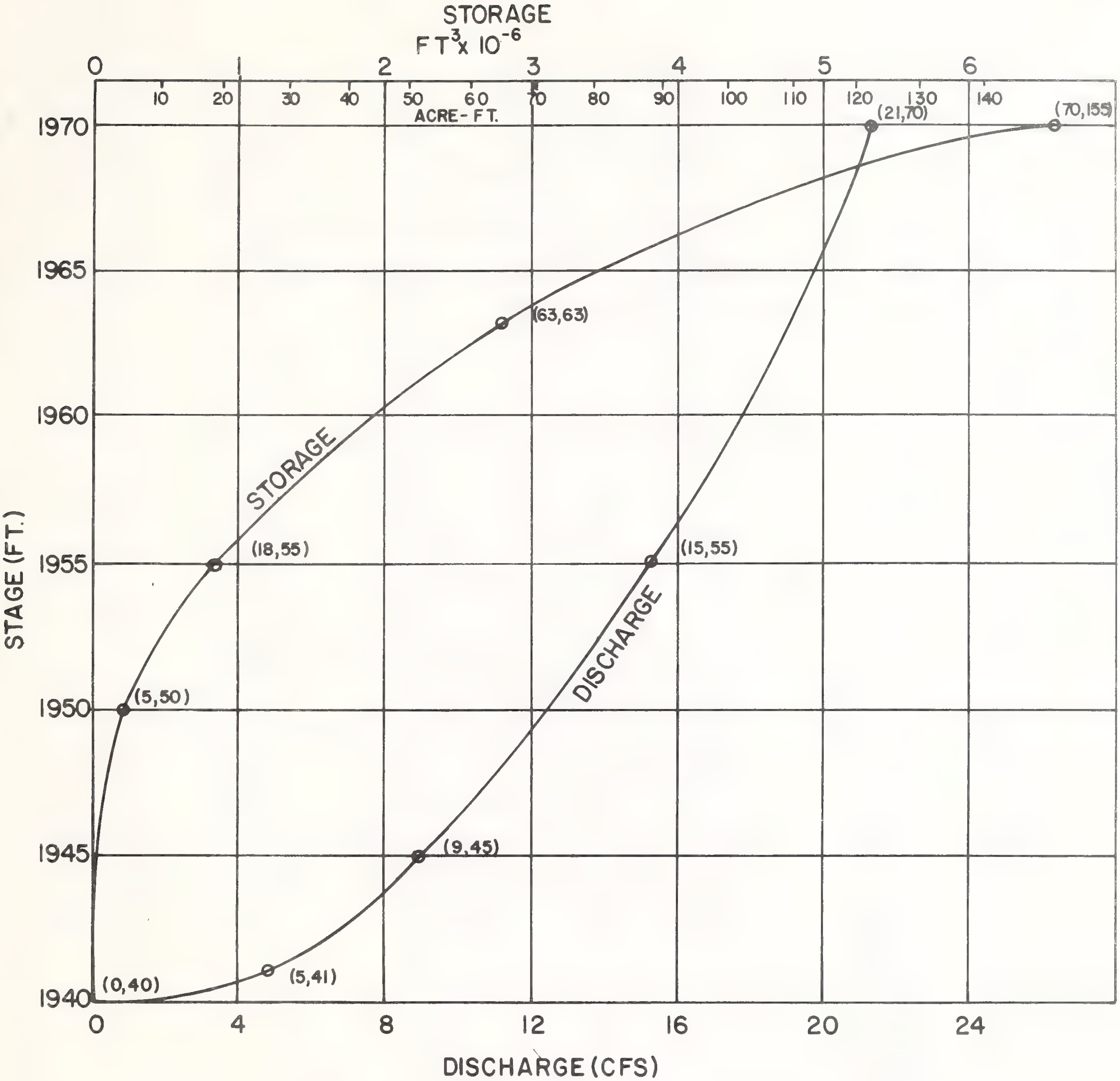


Figure 4.69 - STAGE-STORAGE AND STAGE DISCHARGE CURVES

```
     edit fld ipli
EDIT run
ENTER THE NO. OF INFLOW POINTS
?    11
ENTER THE INFLOW RATE(CFS) ORDINATES
?    0 100 300 250 140 80 50 30 15 7 0
ENTER THE NO. OF STAGE-STORAGE & DISCHARGE PTS. RESP.
?    5 5
ENTER (STAGE(FT),STORAGE(ACRE-FT)) COORDINATES IN PAIRS
?    40 0 50 5 55 18 63 63 70 155
ENTER (STAGE(FT),DISCHARGE(CFS)) COORDINATES IN PAIRS
?    40 0 41 5 45 9 50 15 70 21
ENTER THE TIME INCREMENT IN HOURS
?    .5
```

F L O O D   R O U T I N G

TIME (HRS.)	STAGE (FT.)	INFLOW (CFS)	OUTFLOW (CFS)	STORAGE (ACRE-FT)
0.00	40.00	0.00	0.00	0.00
0.50	43.75	100.00	7.75	1.88
1.00	51.72	300.00	15.52	9.47
1.50	55.37	250.00	16.61	20.11
2.00	56.75	140.00	17.02	27.84
2.50	57.50	80.00	17.25	32.06
3.00	57.75	50.00	17.32	33.47
3.50	58.00	30.00	17.40	34.88
4.00	58.00	15.00	17.40	34.88
4.50	58.00	7.00	17.40	34.88
5.00	58.00	0.00	17.40	34.88
5.50	57.75	0.00	17.32	33.47
6.00	57.62	0.00	17.29	32.77
6.50	57.50	0.00	17.25	32.06
7.00	57.37	0.00	17.21	31.36
7.50	57.25	0.00	17.17	30.66
8.00	57.12	0.00	17.14	29.95
8.50	57.00	0.00	17.10	29.25
9.00	56.87	0.00	17.06	28.55
9.50	56.75	0.00	17.02	27.84
10.00	56.62	0.00	16.99	27.14
10.50	56.50	0.00	16.95	26.44
11.00	56.37	0.00	16.91	25.73
11.50	56.25	0.00	16.87	25.03
12.00	56.12	0.00	16.84	24.33
12.50	56.00	0.00	16.80	23.63
13.00	55.87	0.00	16.76	22.92
13.50	55.75	0.00	16.72	22.22
14.00	55.62	0.00	16.69	21.52
14.50	55.50	0.00	16.65	20.81
15.00	55.37	0.00	16.61	20.11
15.50	55.25	0.00	16.57	19.41
16.00	55.12	0.00	16.54	18.70
16.50	55.06	0.00	16.52	18.35
17.00	54.84	0.00	16.45	17.59
17.50	54.69	0.00	16.41	17.19
18.00	54.38	0.00	16.31	16.38

High Stage

Maximum Discharge

18.50	54.06	0.00	16.22	15.56
19.00	53.75	0.00	16.13	14.75
19.50	53.44	0.00	16.03	13.94
20.00	53.13	0.00	15.94	13.13
20.50	52.81	0.00	15.84	12.31
21.00	52.50	0.00	15.75	11.50
21.50	52.19	0.00	15.66	10.69
22.00	51.88	0.00	15.56	9.88
22.50	51.56	0.00	15.47	9.06
23.00	51.33	0.00	15.40	8.45
23.50	51.09	0.00	15.33	7.84
24.00	50.86	0.00	15.26	7.23
24.50	50.63	0.00	15.19	6.63
25.00	50.39	0.00	15.12	6.02
25.50	50.16	0.00	15.05	5.41
26.00	49.69	0.00	14.63	4.84
26.50	48.44	0.00	13.13	4.22
27.00	47.50	0.00	12.00	3.75
27.50	46.56	0.00	10.88	3.28
28.00	45.63	0.00	9.75	2.81
28.50	44.88	0.00	8.88	2.44
29.00	44.25	0.00	8.25	2.13
29.50	43.63	0.00	7.63	1.81
30.00	43.00	0.00	7.00	1.50
30.50	42.44	0.00	6.44	1.22
31.00	41.94	0.00	5.94	0.97
31.50	41.44	0.00	5.44	0.72
32.00	41.02	0.00	5.02	0.51
32.50	40.67	0.00	3.36	0.34
33.00	40.44	0.00	2.19	0.22
33.50	40.29	0.00	1.45	0.14
34.00	40.19	0.00	0.94	0.09
34.50	40.13	0.00	0.63	0.06

EDIT

Detention Time

References

Linsley, R.K. and J. B. Franzini, Water-Resources Engineering, McGraw-Hill Book Company, New York, 1964.

SECTION 4.10
IRRIGATION

4.10 IRRIGATION

Introduction

The purpose of this section is to acquaint the designer with the various types of structures encountered in irrigation systems, to present procedures for determining design flows, and to present general guidelines for canal design. The design of irrigation facilities can usually be accomplished with information presented in previous sections. Sections 4.1, "Culvert", 4.2, "Bridges", 4.6, "Open Channel Flow", and 4.8, "Erosion Control" must be referred to for design of the respective facilities.

4.10.1 GENERAL GUIDELINES

Most of the irrigation ditches, crossings, and structures that must be rebuilt during highway construction must be reviewed and approved by the landowner, ditch company or agency that owns and controls the irrigation system. Therefore, the first step in the design of any irrigation facility is to contact the owner or operator and determine their requirements. These requirements should include design criteria such as typical sections, flow, slope, and freeboard on all main canals and laterals that may be disrupted. Information concerning the need for trash racks, type of pipe preferred, improvements, and recommendations on canal linings if applicable. This information along with the data submitted by field survey personnel (see Section 2, "Surveys") is generally sufficient for design. In situations where the above information is unavailable, the following general guidelines will help design the system.

For unlined canals, the bottom width to depth ratio usually ranges from 2:1 for small channels to 5:1 for canals with capacities of about 500 c.f.s. Canals with hard - surface - linings usually have bottom width to depth ratios of 1:1 for small channels to 2:1 for large channels.

The steepest satisfactory side slopes are 2:1 for an unlined channel and 1.5:1 for channels with hard - surface - linings.

Freeboard ranges from 1.0 feet for small unlined ditches, up to 3.0 feet for unlined canals carrying 500 c.f.s. The freeboard for hard - surface - lined canals ranges from 6 inches for small laterals to 2 feet or more for larger canals Additional freeboard above the top of the lining of 6 inches for small laterals to 2 feet for large canals should also be provided.

The radius of curvature to the canal centerline for unlined canals should be at least three to seven times the water surface width with the larger ratios for the larger capacities. A lined canal should have a minimum radius of curvature of three times the water surface width.

Velocities in unlined canals ordinarily vary from 1.0 to 3.5 feet per second while velocities from 1.0 to 8.0 feet per second are common for canals with hard-surface.

The following values of Manning's roughness coefficient shall be used for new construction. When evaluating the capacity of an existing system, these values may be adjusted to compensate for varying site conditions.

unlined ($Q < 100$ cfs)	0.025
unlined ($Q > 100$ cfs)	0.0225
Portland Cement concrete lining	0.015
shotcrete lining	0.017
asphaltic concrete lining	0.014
soil cement lining	0.016

When aquatic growths are expected, an increase in n should be considered in the design to accommodate the increased flow resistance. Increases should be as much as 30% in canals heavily infested with filamentous green algae.

The bottom of a canal with a hard - surface - lining should be at least 3 feet above the water table to prevent damage from freezing and thawing.

4.10.2 FLOW CAPACITIES

The most important criteria in designing an irrigation canal or structure is the capacity. Without proper data on capacities, the design procedure becomes more involved and the results less certain.

Several methods of determining a design flow can be used. Three such methods are listed below in order of preference.

1. Data submitted from owner
2. Capacities of existing systems
3. One c.f.s. for every 40 acres to be irrigated

The third method should be used only as a last resort. Information contained in the "Water Resources Survey" prepared by the Montana Department of Natural Resources, Water Resources Division is sometimes helpful in determining existing capacities and water rights.

4.10.3 IRRIGATION STRUCTURES

The following paragraphs deal with commonly used irrigation structures.

Checks - Checks are used to control the flow beyond the structure or to maintain a certain water depth above the structure. Checks may be a separate structure or combined with drops, chutes, or turnouts. Checks are often desirable to prevent racing and scouring upstream from chutes and drops.

Check structures generally use radial gates, slide gates, or flash boards to control the amount of water passing the structure. Radial gates are generally used in very large structures. One or more slide gates are often used for smaller structures. Flash boards are most commonly used for checks with capacities less than 50 cfs, and where operational changes are infrequent. Flash boards should not be used in openings greater than 5 feet wide or with depths over 6 feet. For the greater depths, the guides should be sloped 1:4 downstream from the bottom to top of the flash board guides.

The need for safety provisions should be considered on all checks.

A head loss equal to .5 times the difference in velocity heads between the check opening and the upstream canal section is usually adequate. About 3.5 feet per second is the maximum velocity through check structures using flash boards, owing to difficulty in operation, whereas a velocity of 5 feet per second is not objectionable through most structures using gates.

Flumes - Flumes are used to convey water along steep hillsides, across depressions, and where restricted right-of-way or other reasons make the construction of canal banks undesirable or unreasonable. Rectangular concrete flumes are the most common with steel pipe used in some installations. Concrete footings with concrete or steel supports are generally used for elevated flumes.

To attain economy in the use of materials, it is desirable to give the flume sufficient slope to permit a reduction in cross section. However, the

slope should not be great enough to permit the flow to approach critical depth. Transitions must be used to reduce head losses at entrances to flumes.

Freeboard in a flume should be correlated with that in the adjacent canal; it should be set to overtop before the canal banks or be higher than the adjacent banks, whichever will give the least damage should overtopping occur.

Chutes and Drops - Chutes and drops are structures commonly used to convey water to a lower elevation so as to avoid excessive velocities and erosion of canal banks and beds. Chutes and drops are usually rectangular in cross section; however, trapezoidal cross sections can occasionally be used for small capacities. Chutes and drops are usually constructed of concrete or steel.

Chutes may be considered in three sections: (1) the section of transition and accelerating velocity; (2) the chute channel with high velocities; and (3) the stilling basin. In the first section, the velocity is increased from the canal velocity up to 20 or more feet per second and the cross section area of the water is proportionately decreased. Checks are often combined with the inlet to prevent racing of the water upstream from the inlet. The second section, the chute channel, will usually consist of a length of channel with its grade following the general configuration of the original ground surface, and a short steep section leading to the stilling pond. The third section, the stilling basin, is required to dissipate the kinetic energy. Stilling basins are discussed in Section 4.8, "Erosion Control". A minimum of 15 inches of freeboard, measured normal to the chute slope, should be provided.

Vertical drops are often used in canals and laterals to dissipate a few feet of energy. The maximum vertical drop in water surface is about 3 feet for flows less than 70 c.f.s. and 1.5 feet for flows larger than 70 c.f.s., except where hard - surface lining or paving is provided downstream from the structure. The maximum drop in a concrete lined canal would normally be about 6 feet. A check structure should normally be combined with a vertical drop to prevent

drawdown and scour upstream from the structure.

Turnouts - Turnouts divert water from a main water supply channel to a smaller channel or a farm irrigation ditch. The large turnouts are usually designed as open channels with a bridge, while the smaller structures usually have a covered conduit. Radial or slide gates are generally used to control the amount of water diverted through the turnouts. The structure must be large enough and set low enough to carry the required flow from a checked water surface in the main channel. Water measurement structures are often required at turnouts.

Weirs - The most commonly used structure for measuring irrigation flows is the weir. Weirs are relatively accurate, simple, and easy to construct, and quite durable. However, they do require considerable fall of water surface.

The weir should be designed so the head over the weir is greater than one-third the weir length and the depth over the weir is greater than .2 feet or the accuracy will be reduced. The weir crest should be sharp edged, and the stream over the weir should have free fall with the admission of air under the stream. A water measurement manual or a hydraulics text book should be consulted to determine the weir dimensions.

REFERENCES

Chow, V.T., Open-Channel Hydraulics, McGraw-Hill Book Company, New York, 1959.

Canals and Related Structures, Design Standards No. 3, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, 1967.

Water Resources Surveys, Part I and II, a county by county report published by the Water Resources Division, Montana Department of Natural Resources and Conservation, Helena, Montana.